EXPERIMENTAL INVESTIGATION OF THE TYING CAPACITY OF WEB CLEAT CONNECTIONS IN FIRE

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INTRODUCTION

Structural steel connections have been extensively investigated over the past three decades to determine their moment-rotation characteristics. However, the importance of tying capacity had been realized even earlier, since the explosion at Ronan Point [1] in 1968 caused progressive collapse of a large part of the building; as a result of this incident the Building Regulations [2] in the UK were amended to require positive tying capacity between key elements. Later, the BS5950 code [3] introduced a clause requiring connections to have minimum tying capacities, but without solid support from research evidence. The UK SCI/BCSA design guidance [4] checks the tying capacity as a separate action, whereas in reality a combination of tying force, shear force and moment always exists. Guravich [5] investigated the resistance of connections in combined shear and tension. For individual bolts, the resistance to tying force may be affected by the co-existence of other forces. For a complete bolted connection the co-existence of other actions can prevent a uniform distribution of the tying force to each bolt, causing an “unzipping” mode of failure, and significantly reducing the tying capacity.

In design for fire resistance, adoption of performance-based design principles in more and more countries means that structures are now treated integrally in structural fire safety design. Connections, as the key components which tie structural members together, are important in maintaining structural integrity and preventing progressive collapse. However, evidence from the collapse of the WTC buildings [6] and full-scale fire tests at Cardington [7] have shown that connections may often be vulnerable to fracture. So far, only very limited research has been done on the performance of connections at elevated temperatures, most of which has concentrated on endplate connections, and has mainly been confined to moment-rotation curves. A further complexity which affects the investigation of connection behaviour in fire is the fact that the interactions between structural members cause continuous changes in the forces and moments taken by the connections. The properties of the connection can also affect its internal forces during a fire.

The Universities of Sheffield and Manchester have conducted a joint research programme with the aim of investigating the capacity and ductility of steel connections at elevated temperatures. A recent trend in the design of composite framed structure has been to fire-protect beams on the main column grid, while leaving other beams unprotected. The protected beams will eventually deflect considerably under the combined effect of high steel temperatures and enhanced loading, shed from the unprotected members, and will impose high tying forces on their connections. In non-composite steel construction, the beams deflect at high temperatures and experience catenary tension, which is transferred to the supporting structure through the connections at the beam ends. Previous tests [8] have shown that connections can be subjected to tying forces varying from 0.65 to 1.6 times the shear force at high temperatures. Various levels of moment may be transferred through the connections, depending mainly on the connection type. Hence, the current investigation adopted a test setup in which the connections were subjected to a combination of tension and shear forces. Moments were generated at the connection due to the lever arm of the applied force. In total, four types of connection were studied; flush endplates, flexible endplates, fin plates and web cleats. This paper reports the test results on web cleat connections.

TEST SETUP

A detailed description of the test setup and test measurements has been given in a previous paper [9], so only a brief overview will be given here. The tests were performed in an electrically heated oven of 1.0m^3 internal capacity, as shown in Figure 1. The specimens were heated slowly to the specified temperature, and then loaded to failure at constant temperature. A special loading system was designed to allow very large rotation of the tested connection. It includes three link bars, one end of each connected to a central pin, with the other ends respectively connected to the jack, the specimen and a fixed hinge. When the head of the jack moves downward, it applies a tensile force to the end of the specimen through the action of the linkage. The loading jack was displacement-controlled. The applied load was measured from strain-gauges attached to the bars. The deformations of the connection were measured using a digital camera facing the connection through a glass window in the oven wall.

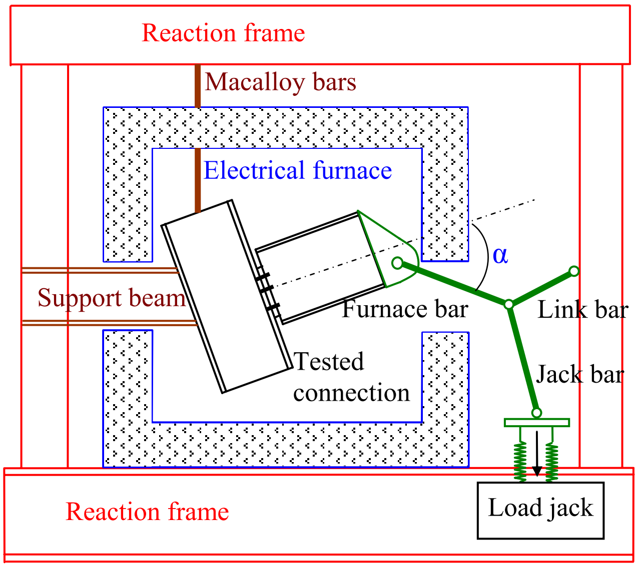


Fig. 1. Test setup

A UC254×89 section was used for the column, and the beam specimens were all UB305×165×40. Figure 2 shows the details of the connection, which is typical of practical designs in accordance with UK design recommendations [4]. Twelve specimens used equal angles of L90×8 section, shown in Figure 2. Two specimens were designed using unequal angles of L150×90×10 section. In this case the beam web was strengthened by welding a 10mm thick plate to each side. Therefore, in the connected region, the total thickness of the beam web was 26mm. A custom-made connector was bolted to the end of the beam, and the load from the tie-bar was applied to this connector through a pin.

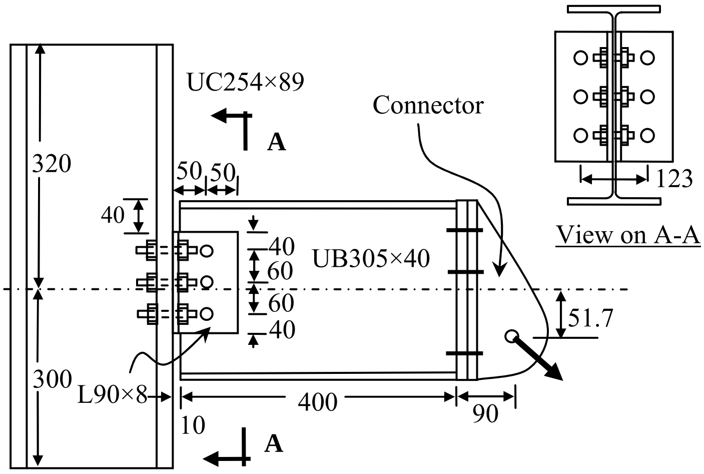


Fig. 2. Details of the test specimen

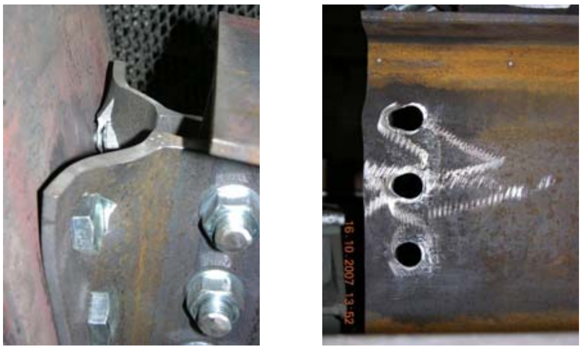
TEST RESULTS

A summary of the test results is given in Table 1. The second column gives the uniform temperature of the specimen. The third column is the initial loading angle α. Connections were designed to be tested at three different combinations of the shear and tying forces, corresponding to different angles α in Figure 1. This angle α is actually that between the axis of the steel beam and the furnace bar. Its value is the sum of their respective inclinations to the horizontal axis, which were measured using two cameras. Three nominal initial angles α, of 55°, 45° and 35°, were chosen. However, it was not possible to set these exactly at the start of any test, and so the exact initial value is given in the fourth column. During each test the angle α changed progressively from its initial value, the degree of variation depending on the exact geometry of the loading system. Its value at the end of each test is shown in the fifth column of Table 1. The final two columns show the maximum resultant applied force and the connection rotation at maximum resistance.

Table 1. List of test results

Specimen geometry	Temperature (°C)	Load angle (Degree)	initial α (Degree)	ending α (Degree)	Force (kN)	Rotation (Degree)
1. 3-8.8-20	20	55	55.0	34.4	186.34	16.57
2. 3-8.8-20	450	55	55.8	43.5	93.74	9.39
3. 3-8.8-20	550	55	56.0	42.2	52.91	10.52
4. 3-8.8-20	650	55	56.5	34.4	25.70	14.15
5. 3-8.8-20	20	45	45.7	32.0	212.54	17.12
6. 3-8.8-20	450	45	46.7	37.3	99.42	10.29
7. 3-8.8-20	550	45	47.0	36.8	56.35	11.53
8. 3-8.8-20	650	45	48.1	34.5	28.18	15.94
9. 3-8.8-20	20	35	37.4	21.2	243.17	16.71
10. 3-8.8-20	450	35	41.1	29.1	112.85	10.75
11. 3-8.8-20	550	35	41.4	26.6	61.21	12.56
12. 3-8.8-20	650	35	40.9	21.6	31.57	14.86
13. 6-8.8-20	550	35	40.2	27.2	85.01	10.95
14. 6-8.8-20	550	55	55.7	41.0	66.78	9.19

The failure mode was observed to change with temperature, but to remain the same for different loading angles at each temperature. At 20°C, a typical failure of the connection is shown in Figure 3. The failure was initiated by the bolt head punching through the leg of the cleat on the column face. A block-shear fracture was then developed from the bolt hole towards the top of cleat. This fracture happened at a very high rotation. The two cleats had by that stage undergone a significant amount of deformation. At failure, the beam web could be seen to have undergone obvious bearing deformation around the bolt holes. All the bolts remained undamaged.



a. Failed connection b. Beam web deformation

Figure 3 Failure mode at ambient temperature

At 450°C and 550°C, fracture happened close to the heel of the cleats adjacent to the beam web, as shown in Figure 4. Before fracture, deformations of the two cleats were lower than that at ambient temperature. Bolt holes on the beam web had noticeable bearing deformations, although lower than those at ambient temperature. Deformations of the three bolts connecting the two cleats to the beam web are shown in Figure 4. These can be seen to have various degrees of bearing deformation. Bolts in the top two rows connecting the cleats to the column flange experienced bending deformations close to the bolt head, while the others remained straight.

At 650°C, no web cleat fracture was observed. The connections failed by shearing the bolts connecting the cleats to the beam web. It was observed that, as the connection rotation increased, the bearing deformations of these bolts were increased until

a. Failed connection b. Beam web

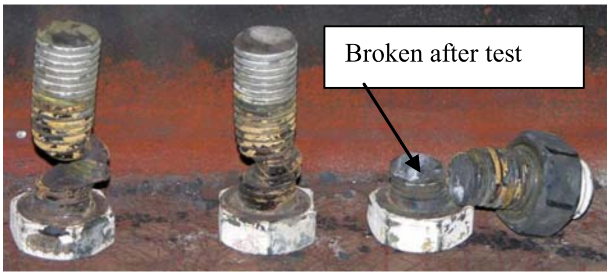
c. The three bolts through the beam web

Figure 4 Failure mode at 450°C and 550°C complete fracture of the bolts in double shear occurred. Figure 5 shows the deformation of the connection, loaded at α = 45°; this test was stopped immediately after the resistance started to drop. For the other two loading angles, for which the tests were continued further, the top two bolts were completely sheared through and the bottom bolt had acquired significant bearing deformation. The bolts connecting the two cleats to the column flange had experienced significant tensile and bending deformations. The top two bolts showed necking close to the bolt head.

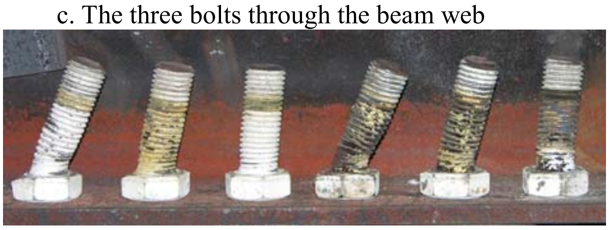
Figure 6 shows the force-rotation relationships for the first 12 tests. Assuming that the bolts were initially placed in the middle of the 22mm clearance holes, it takes an initial rotation of about 2° for the top and bottom bolts to come into contact with the edges of the holes. However, no significant applied force increase was observed in any of the tests because of this contact. It was not until the lower flange of the beam made contact with the column flange, at around 4°, that the total applied force started to increase rapidly in all cases. With the gradual deformation of the web cleats, the applied forces then entered a prolonged rising phase before reaching a maximum. The ductility, defined as

a. Failed connection

b. Beam web

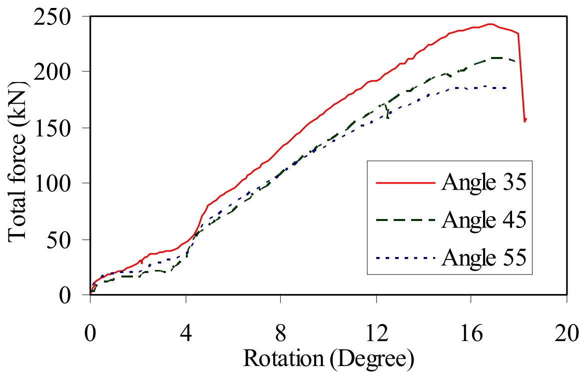


c. The three bolts through the beam web

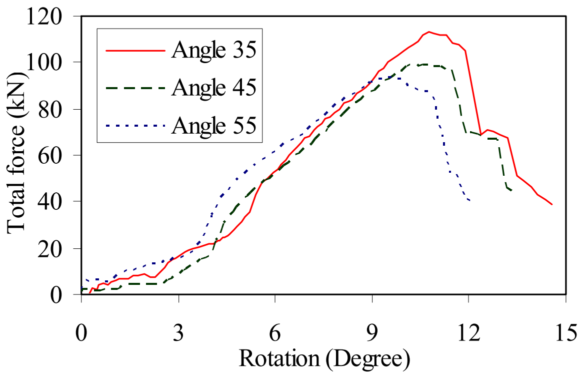


d. The six bolts connected to the column flange

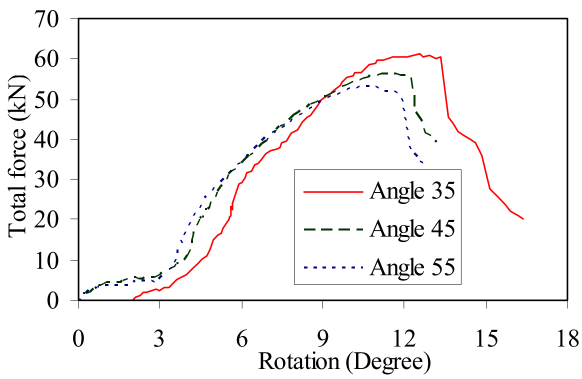
Figure 5 Failure mode at 650°C



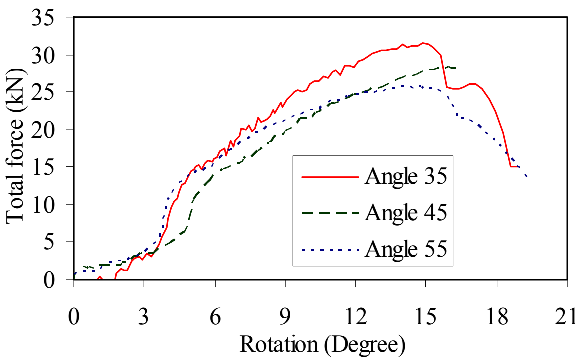
a. at 20°C



b. at 450°



c. at 550°



d. at 650°C

Figure 6 The force-rotation relationships of the tested connections

the angle of rotation achieved at the maximum force, is determined by the failure mode in a particular test. Fracture of the web cleat close to its heel at 450°C and 550°C obviously reduces the ductility.
In all the tests, bolts are fully-threaded Grade 8.8 bolts with an average tensile resistance of 224kN. Steels are classified as S275, with a yield strength of 340 N/mm^2 for the angle and 356 N/mm^2 for the beam. Their properties at elevated temperatures were not tested directly. The connection tests were performed a very slow deflection rate and at constant temperature. After being heated to the desired temperature, the specimen was progressively loaded to fracture in about 90 minutes. This type of test is classified as steady-state test, in which the elevated temperature properties of steel could differ significantly from those obtained in transient tests such as that suggested by EC3: Part 1.2 [10]. Renner [11] performed an investigation of the properties of S275 structural steel at elevated temperatures under steady-state test conditions. A group of bolts with similar properties was tested at elevated temperatures up to 600°C by Hu *et al.* [12].
The reduction of the connection resistance (the maximum applied force) with temperature is shown in Figure 7. The specimens loaded at different angles basically follow the same force reduction curve. For comparison, Figure 7 also shows the reduction of the yield and ultimate strengths for normal steel, and the strength of bolts, according to EC3: Part 1.2, reduction of the ultimate strength for normal steel according to Renner, and the reduction of the tensile resistance for Grade 8.8 bolts according to Hu. The failures observed in the connection tests were due to combinations of local failures of steel sections and bolts. As Renner's and Hu's test data are both obtained from steady-state fire test, they should correlate better than the Eurocode strength reduction curves with the current connection tests. It can be seen from Figure 7 that reduction of the ultimate strength for normal steel from Renner, and reduction of the bolt strength from Hu, are very close, and the reduction of the tested connection resistance basically follows these two curves.

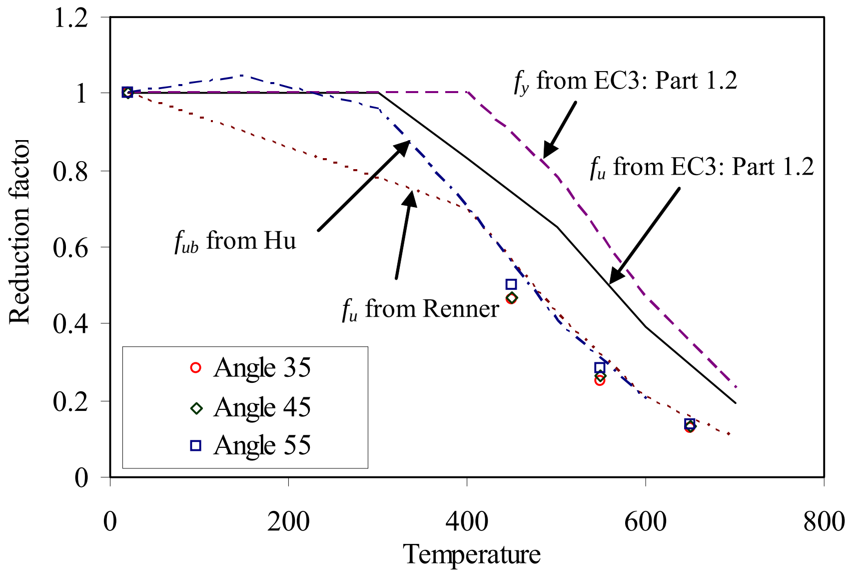


Figure 7 Reduction of the connection resistances with temperature

CONCLUSIONS

This paper reports on an investigation of the robustness of web cleat connections under tying force at elevated temperatures. The two key parameters studied are the tying capacity of the connection in the presence of other actions, and the rotational capacity. Tests were performed on web cleat connections at various temperatures, and subjected to different combinations of tension and shear loading. Tests results showed that the tying capacity decreased rapidly with increase of temperature, and that the connection had little residual resistance at 650°C.
Web cleat connections are capable of failing in a number of ways. The failure mode appears not to be sensitive to the load combinations, but is dependent on the temperature. Fracture of the web cleat close to its heel, and double shear of the bolts through the beam web are the two critical failure modes at elevated temperatures. In general, web cleat connections have extremely high rotational capacity compared with alternative types [13].

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