Fire Behaviour of Structural Steel – Code vs. Tests

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Abstract. This article shows part of the results of a complex research on the behaviour of steel beam-to-column end-plate connections under the fire action. In the paper, a parallel between a computation made with the help of a computer model, based on the prescriptions of the European design code EN 1993 and experimental tests conducted on real scale substructures is presented. The main goal of this work is to find the difference, in terms of strength and mode of failure, between the European design code and the experimental situation in the testing facility.

1. Introduction

An experimental testing programme was done in 2014-2015 at the Technical University of Cluj-Napoca, Romania, Faculty of Civil Engineering Testing Facility, on 7 real scale steel beam-connection-column substructures. The global programme was aimed to find the behaviour of end-plate connections under normal load at high temperature and also the behaviour of the ensemble under the post-earthquake fire action.

Before designing and executing the real scale specimens, multiple simulations were performed by help of computer programs. Firstly, we designed a structure having 4 stories and made up of steel frames. The nodes were realized by end-plate bolted connections. Such a connection was then designed by applying the prescriptions according the European design code EN 1993 [1]. The second stage consisted in verification of the connection at surrounding temperature and also at high temperature – by modifying the properties of the material and bolts according to the design code [1]. The force at which the connection failed and the failure mode were computed at 20°, 200°, 400°, 600°, 800° Celsius by applying [1]. After that, nonlinear fire simulations on the entire sub-structure were performed and we passed to the execution and test of the specimens.

2. Setting up the test stand

The main idea when designing the test stand was to assure that the realized links are according to the links that are going to be used in the numerical simulation, according to Petrina and Muntean [2]: ‘The connection had to be fixed in order not to permit rotations of the ensemble and also not to permit displacements in a direction normal to the plane formed by the axes of the beam and of the column’. In the author’s opinion, these main directions assure the conditions inside a real steel frames structure under normal loading.

The furnace was placed such in a way to provide the gas flame exactly on the connection as shown in Petrina and Muntean [2]. The material is reflective thermo bricks. In figure 1, one may see the position detail of the furnace. Studying different articles on fire tests like Bursi [3], Puccinotti et. al. [4], we decided to follow these main characteristics for the furnace: “to have sufficient power, to have a good thermal insulation, to be constructible around the node, to be removable after each test” [2] (mechanical,
fire), so no binder was used between bricks and to permit beam movement up and down in order to apply the testing procedure described in ECCS document [5]. We avoided to use an electrical furnace for this magnitude of structure, due to limited power of such a type. During tests, our furnace with methane gas burner had enough power and it was used at a quarter of its power. Regarding the thermal insulation of the furnace, this was assured by the thermos bricks, the temperature remained constant (in a range of 15°C) during the mechanic action. The reinforced concrete precast plates that assured the top of the furnace were executed with a 12.5 cm layer of bricks on the lower part exposed to fire as explained in Petrina and Muntean [2]. The zone in which the beam exits the furnace was provided with mineral fire resistant wool which permitted the cyclic action to be done exactly before fire test. Constructing the furnace with no binder permitted the removal of the furnace after each test, in order to change the specimen in the stand.

The originality aspects of setting up this stand for fire and post-earthquake fire tests were about the design of beam, node, column ensemble; design of the test stand (sections and dimensions of steel elements for the prior computed and expected internal forces); the way to test the specimen, the position of the specimen and the entire programme management; the cyclic action on the specimen by using only monotonic pressure devices and displacement translator; the design of a removable furnace around the connection, with a low cost and using traditional materials, which permits immediate fire test after the cyclic action; the design of a cooling mode to protect gas burner’s blower when stopping fire.

![Image](image_url)

**Figure 1.** Entire set-up ready for mechanic + fire test according to Petrina and Muntean [2]

The tensions in the node area, magnitude of applied force on the cantilever, the displacement of the free end of the cantilever and the temperature of the steel in the node area, were the main data we wanted to have registered during tests. Steel test stand, furnace made by bricks with no binder, having a 155kW gas burner, thermocouple, one press having two cylinders for mechanic action, displacement and force translators were arranged as in figure 1. The equipment we used was: HBM - Quantum C - MX 1615 + software Catman according to Petrina [6] arranged like in figure 1.

The loads were applied by 1 press with 2 cylinders and the magnitude of the forces and displacements was measured by specific devices. The burner is of type Weishaupt which works on methane gas, all arranged like in Figure 1.

3. Specimen verification according to the design code EN 1993 [1]

3.1. Description of the studied structure and steel properties

The column is made up by an H type compound profile having the flanges of thickness 15 mm and web 10mm and the beam is of I type compound profile with the thickness of the flanges of 15 mm and the web 8mm. The bolts are of type 10.9 with controlled tense. The connection is a steel beam-to-column bolted connection. The column is 3.0m high and the cantilever beam has a length of 2.0 m. The type of the material used is structural steel S235JR.
There are 12 M20 bolts for each connection, having the length of 70 mm (see Figure 2). The column is stiffened by 15mm and 20mm steel plates like in the model above. The steel S235JR properties at 20 Celsius degrees: \( E = 210 \, 000 \, \text{N/mm}^2; \) \( G = 81 \, 000 \, \text{N/mm}^2; \) Poisson’s coefficient \( \nu = 0.3; \) \( \alpha = 12 \times 10^{-6}; \) \( f_y = 235\,\text{Mpa}; \) bolt strength \( F_b = 176.4 \, \text{kN}. \)

3.2. Verification at room temperature, at 40 °C and at 600 °C
According to the design code prescriptions, formulas and verifications, at 20°C, the connection resists at an applied load on the free end of the cantilever equal to 125kN. For verification of the connection at high temperature, the steel and bolts properties were reduced according to EN 1993 [1]. According to the results of the verification, the connection at 400 °C, resists at a maximum force applied on the free end of the cantilever equal to 105kN and at 600 °C, it resists at a maximum force equal to 51 kN.

3.3 Failure modes at 20 °C, 400 °C and 600 °C
According to the verifications, by applying prescriptions according to [1], the studied connection: At 20°C, failure occurs by column web panel shear; that is when the sum of forces taken by the bolt rows above the verified bolt row is greater than the web panel shear resistance. In this case only the first three rows of bolts are carrying loads, the remaining bolts are inactive even if the 4th and the 5th rows are above the center of rotation. At 400°C, all rows of bolts situated above the center of rotation of the connection (the first 5 rows counted from top to bottom) are carrying loads. In this case, the failure of the connection is due to the rupture of the first row of bolts. At 600°C, failure occurs by column web panel shear. The first 4 rows of bolts are carrying loads in this case, the 5th doesn't, because the sum of forces taken by the first 4 rows is greater than the shear capacity of the panel web. The 6th row of bolts is inactive because it is situated below the center of rotation of the connection.

4. Maximum force and failure modes during experimental tests
4.1. Behaviour of the specimen at 20°C
During this test the free end of the cantilever was subjected to increasing load while the temperature remained 20 degrees. The response of the connection in terms of force undertaken by the connection is given in the chart below. In figure 3, one may see that the maximum force was around 195 kN.
Figure 3. Time – force curve for room temperature test

The failure occurs by the rupture of the 1\textsuperscript{st} and then of the 2\textsuperscript{nd} row of bolts. The rupture is associated by a bending of the column flange near the 3\textsuperscript{rd} row of bolts as in figure 4.

Figure 4. Failure of the connection at room temperature test

4.2. Behaviour of the specimen at 400 °C

The connection was heated at 400 degrees and pressed on the free end of the cantilever with increasing force. The failure mode may be seen in the figure 5.

The maximum force is equal to 101 kN when the 1\textsuperscript{st} row of bolts breaks. After that, the 2\textsuperscript{nd} row of bolts breaks at a force value around 75 kN. The failure is sudden and accompanied (like at 20 degrees) by the bend of the column flange near the 3\textsuperscript{rd} row of bolts.
4.3. Behaviour of the specimen at 600 °C

The connection was heated at 600°C; the response of the connection in terms of force and displacement is in figure 6. The maximum attained force that was supported by the connection at a temperature of 600 degrees, was 48 kN.

The mode of failure is not sudden, like in the previous two cases. This may be observed in the upper graph and also in figure 7. The failure, in this case, is by buckling of the lower flange of the beam accompanied by the yield of the first two rows of bolts until complete fracture.

5. Conclusions

A very good correlation between verifications according design code and behaviour in experimental situation was obtained. The results were also in accordance with other studies like Faggiano and Mazzolani [7]. The design of these types of connections according to the design code [1] offers a plus of resistance: around 52% of the designed value, at room temperature. In the theoretical case, the maximum force at 20 degrees was 125 kN, but during experimental test it was around 180-200 kN. At 400 degrees, instead, the designed maximum force (105 kN) is very similar to the experimental maximum resulted force (101kN), so at this temperature, the design according to EN 1993 [1] fits perfectly the real behaviour. At 600 degrees the designed maximum force (51 kN) is like at 400 degrees, very similar to the experimental resulted force (which was 48 kN).
The conclusions in terms of failure modes are the following: At 20 degrees, according to the formulas and tables from EN1993 [1], we get that the studied connection fails by the shear of the column web panel, but the experimental behaviour shows that, in reality, the connection fails by the sudden consecutive rupture of the first two rows of bolts. At 400 degrees, according to the design code [1], the failure of the connection is by the rupture of the first row of bolts. In this case, during the experiment, the connection failed also by the sudden rupture of the first row of bolts, continued with the rupture of the second row. We have, in this situation, an exact correlation design code vs. experimental test. At 600 degrees, according to EN1993 [1], the studied connection fails by the shear of the column web panel; in the experimental situation, the fail is by buckling of the lower flange of the beam accompanied by the rupture of the first two rows of bolts in gradual way.

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