Post–Earthquake Fire Tests – Part 2: Failure Modes

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Abstract. During seismic motions, the steel connections are parts of structure that may have permanent damage, changing their strength and stiffness. For this reason, during a post seismic fire, they will work in a different manner, having a different fire resistance. With the help of tests, we want to find the effect of the seismic damage on the fire behavior of the structure. Fire and post-earthquake fire tests were done, together with room temperature tests, for comparison and scale. The main goal was to find the effect of the deterioration due to cyclic action, on the fire behavior of specimens. During earthquake, steel connections may develop a deterioration of strength and stiffness that will lead to a different behavior under fire. In this paper, parallels between different tests are presented. The conclusions are in terms of maximum reached forces and modes of failure in the case of prior deteriorated or new connections subjected to fire.

1. Introduction

This study is following the first part in which the report of post-earthquake fire tests made at the Technical University of Cluj-Napoca, Romania, Faculty of Civil Engineering Testing Facility was presented. Real scale steel connections were created, following prescriptions of the European design code EN 1993 [1]. Some of them were subjected to cyclic action following a special procedure. After the cyclic action, the deteriorated specimens were immediately subjected to fire. New connections were also tested for fire action in order to find differences to the deteriorated case. The main goal was to find the effect of the deterioration due to cyclic action, on the fire behavior of specimens. Such results were obtained by other authors by numerical methods, for example, in Yassin [2].

2. Maximum attained forces during tests

As shown in the first part, the first two tests were done following the procedure proposed by the European Convention for Constructional Steelwork, Technical Working Group 1.3. in “Recommended Testing Procedure for Assessing the behavior of Structural Steel Elements under Cyclic Loads”, documents no. 45, aiming at finding $F_y^+$, $e_y^+$, $F_y^-$, $e_y^-$ (“yield loads in the positive/negative force range, absolute values of displacements in the positive/negative distance range”, according to [3]). During the first test, the maximum attained applied force on the free end of the cantilever on the downwards direction was equal to 186 kN, during second (where an increasing upwards force was applied) the maximum force was equal to 196 kN.

During the third test, the specimen was firstly deteriorated by cyclic action at 20°C, following the previously mentioned procedure, after that, subjected to increasing downwards force until a maximum of 83 kN, where failure of connection occurred.

Fourth test was a post-earthquake fire test at 600°C. The steel beam to column connection was cyclically loaded at 20°C, following the procedure, then heated inside furnace until the steel temperature
reached 600°C. An increasing downwards force was then applied on the free end of the cantilever and the failure occurred at a force magnitude of 41 kN. Fifth test was a fire test at 600°C. The maximum reached force was 48 kN.

The sixth test was a post-earthquake fire test at 400°C. The maximum reached force in this case was 101 kN. The seventh test was a fire test at 400°C. The maximum reached force was surprisingly over 210 kN, more than the forces in case of room temperature tests, according to Petrina [4].

3. Modes of failure during tests
In the first two tests the failure of the connection was by the fracture of the first row of bolts like in figure 1 for test 1 and figure 2 for the test 2. The rupture of bolts was accompanied by the bend of the column’s profile flange between stiffening plates and the bend of the end-plate.

![Figure 1. Failed connection during test 1](image1)

![Figure 2. Failed connection during test 2](image2)

During test 3, the previously deteriorated connection failed by the rupture of the first row of bolts and completely failed by the rupture of the second row of bolts as shown in figure 3. The rupture of bolts was accompanied by the bend of the column’s profile flange between stiffening plates and the bend of the end-plate.

During test 4 (post-earthquake fire test at 600°C), the failure was by the rupture of first two rows of bolts in the same time (all 4 bolts), accompanied by column’s profile’s flange bend and buckling of the beam’s profile’s lower flange like in figure 4. The failure was not sudden like in the previous cases, bolts broke like in figure 5.
During test 5, where a new connection was heated at 600°C and then acted upon by increasing force, the failure occurred by the buckling of the lower flange of the beam, accompanied by the first two rows of bolts failure and also the end-plate bend.

Test 6 was a post-earthquake fire test at 400°C. The complete failure of the connection was like in figure 6, that is, by rupture of first two rows of bolts, accompanied by column’s profile’s flange and connection’s end-plate bend.
During test 6, fire test at 400°C, the first row of bolts broke, after that, the column’s profile’s flange bent and the second row of bolts broke. At that point, the end-plate was also bent.  

4. Parallels and conclusions of tests  
The maximum reached forces and the modes of failure were in accordance with initial numerical simulations and initial computation shown in Petrina [4], following prescriptions of the European design code EN 1993 [1]. They were also in accordance with similar tests presented in Bursi [5] and Puccinotti et al. [6].  
A parallel between the response of the connection at 600°C, in terms of supported force applied on the free end of the cantilever, is shown in the graph from figure 7.
One may notice a difference of maximum 12% between the two curves. A similar behavior of the connection in the two cases (post-earthquake fire test / fire test) was also noticed. The weakest elements of the connection are the bolts. They work fine at 20°C but poor at high temperature.

Another parallel, between connection’s behavior at 400°C in the two cases, is shown in figure 8.

Figure 8: Force-displacement curves at 400°C – parallel between deteriorated/new connection.

The curves show a difference in terms of forces undertaken by the connection, depending on time, of approximately 55%. The shape of the curves is very similar for the two tested cases, having one jump corresponding to the fail of the first row of bolts. This was at an applied force of 110 kN for the prior deteriorated connection and at 218 kN for the second case. The second step in the shape of the curve corresponds to the fail of the second row of bolts at a force of 67 kN for the deteriorated case and 150 kN for the new connection, at 400°C. Here, the same degradation of 55% was noticed.

5. Conclusions
From the experimental testing programme the author deduced that separate seismic design or fire design is not enough for steel structures having similar types of connections. The structure should be analyzed under the combined action. Similar deductions were specified in Faggiano and Mazzolani [7]. That is why the author considers that the post-earthquake fire scenario should be explicitly mentioned in the design codes. The post-earthquake fire action may be considered having a low probability, but it may be characterized by having major consequences. To find the response of a steel structure under a post-earthquake fire, more steps should be done: model the structure by taking into account the material and geometry non-linearity, choose of seismic scenario, a push-over or time-history analysis, evaluation of all fire scenarios, thermal analysis to see the response of each element under fire, structural computation to find stresses and strains. The structural elements and connections are weakened by the inelastic cyclic deformation causing them deterioration of stiffness and strength.

References
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