FIRE PERFORMANCE OF UNDAMAGED AND PRE-DAMAGED WELDED STEEL-CONCRETE COMPOSITE BEAM-TO-COLUMN JOINTS WITH CONCRETE FILLED TUBES

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ABSTRACT
Major earthquakes in urban areas have often been followed by significant conflagrations that have been difficult to control and have resulted in extensive damage to property. Earthquakes then, increase the risk of loss of life if a fire occurs within a building. It is obvious therefore that a seismic-induced fire is a design scenario that should be properly addressed in any performance-based design, in locations where significant earthquakes can occur. In this paper both experimental and numerical results of undamaged and pre-damaged welded steel-concrete composite beam-to-column joints under fire are described as part of a European project aimed at developing fundamental data, design guidelines and prequalification of ductile and fire resistant composite beam-to-column joints. In detail, both the experimental program and fire experimental results are presented and discussed in this paper together with thermal numerical simulations on frames and joints. Both the experimental activity and the numerical work demonstrated the adequacy of the concurrent seismic and joint fire design.

KEYWORDS
Beam-to-column joints, concrete filled tubes, fire, seismic, experimental results.

1. INTRODUCTION

Fire and earthquake are accidental actions and are generally treated in a traditional single-objective design, as independent events (EN 1991-1-2, 2004, EN 1998-1, 2005). Nonetheless, seismic-induced fire is a scenario with high probability of occurrence as evidenced by recent earthquakes in Northridge (1994) and Kobe (1995) (Sekizawa, 2003). Studies of future scenario large-scale earthquakes in San Francisco and Tokyo indicate that the seismic-induced fire will be an important factor in the subsequent damage to property and loss of lives (Wellington Lifelines Group, 2002).
The research presented in this paper attempts to couple fire safety with seismic safety based on the design of full strength composite joints in moment resisting frames in order to guarantee:
1. seismic safety with respect to accidental actions;
2. fire safety where beam-to-column joints are characterized by stiffness deterioration and strength degradation owing to seismic actions.

The present research activity is part of a concluded European research project, devoted to the development of fundamental data, design guidelines and prequalification of two types of fire-resistant composite beam-to-column joints (Bursi et al., 2008c):
1. Type 1 joints, endowed with partially reinforced concrete-encased columns with I-section;
2. Type 2 joints, with concrete filled tubular columns with circular hollow steel section.

For both type 1 and type 2 specimens both interior and exterior joints were considered. In all cases the

connection has been to I-section composite beams. Two different types of slab were analysed. The first type was a conventional composite slab endowed with profiled steel sheeting fixed to the beam through shear studs. The second one was a precast slab with electro-welded lattice girders.
In total six specimens were used for the Type 1 fire experimental program consisting of 2 interior joints with profiled steel sheeting, one exterior joint with profiled steel sheeting, two interior joints with a precast slab and one exterior joint with a precast slab. The Type 2 fire test program also comprised six specimens with an identical distribution between interior and exterior joints and composite and precast slabs. In detail, the experimental program was carried out in two stages: the first stage with static load tests to simulate the damage effect of an earthquake; the second one with fire tests. This paper reports on fire experimental program results together with numerical simulations performed on Type 2 joints as well as on moment resisting frames.

2. DESIGN OF REFERENCE FRAMES UNDER SEISMIC AND FIRE

The actions used in the design of the proposed joints were obtained by means of the analyses carried out on two moment resisting frames having the same structural typology but different slab systems. The composite steel-concrete office-building was endowed with 5 floors with 3.5 m storey height. It was made up by three moment resisting frames placed at the distance of 7.5 m each in the longitudinal direction; while it was braced in the transverse direction. A different distance between the secondary beams was adopted for the two solutions to take into account the different load bearing capacities of the two slab systems as well as the need to avoid propping devices during the construction phase. All slabs were arranged in parallel to main frames as shown in Figure 1.
Two different types of slab were designed and employed. In the first one, the deck was a composite slab with a prefabricated lattice girder, see Figure 2, with slab reinforcements performed by $3+3\phi12$ longitudinal steel bars and by $5+5\phi12@100$ mm plus $8+8\phi16@200$ mm transversal steel bars. A mesh $\phi6@200\text{x}200$ mm completed the slab reinforcement. In the second type of slab a composite slab with profiled steel sheeting was made with the same slab reinforcements. The concrete class was C30/37 while the steel grade S450 was adopted for the reinforcing steel bars.
All connections between steel beams and slabs were full and made by Nelson 19 mm stud connectors with an ultimate tensile strength f_u=450 MPa. In both cases, composite beams were realized with S355 IPE400 steel profiles, while composite columns were realized with 457 mm circular steel tubes with 12 mm thickness; column reinforcements consisted of $8\phi16$ longitudinal steel bars and stirrups $\phi8@150$ mm as shown in Figure 3.
The seismic design of composite beam-to-column joints was conceived to provide both adequate overstrength and stiffness with respect to connected beams, thus forcing the plastic hinges formation in adjacent beams. Joints were detailed by using the component method as shown schematically in Figure 4 (EN 1993-1-8. 2005). Details on seismic analysis and design can be found in a companion paper presented at this conference (Bursi et al., 2008a). The seismic performance of the frames was evaluated by means of non-linear static and dynamic analyses. The corresponding fire design was carried out and the structural fire performance of the complete frames was evaluated by means of the SAFIR program (Franssen, 2002) for different fire scenarios as discussed at length in Section 5.

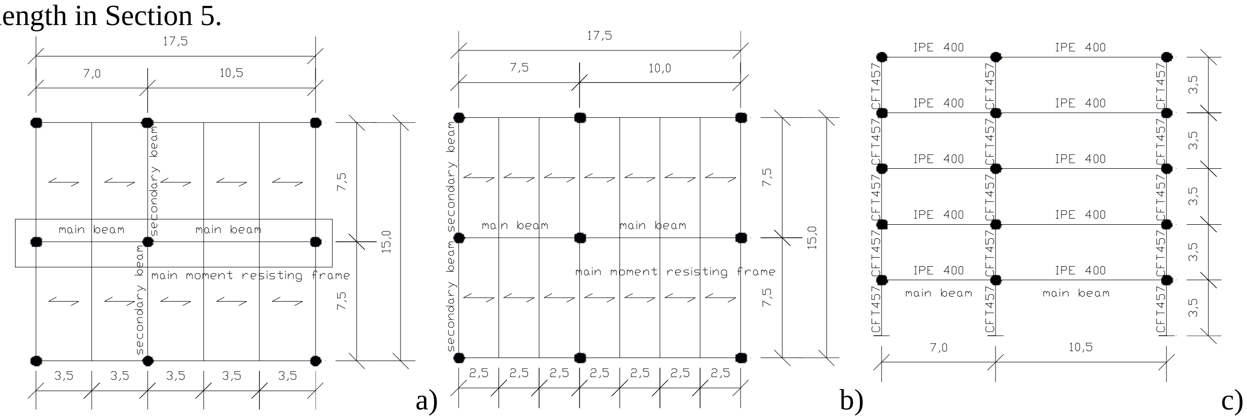


Figure 1 Geometric layout of the reference structures: a) structure with slabs endowed with prefabricated lattice girders; b) structure with slabs endowed with profiled steel sheeting; c) frame elevation

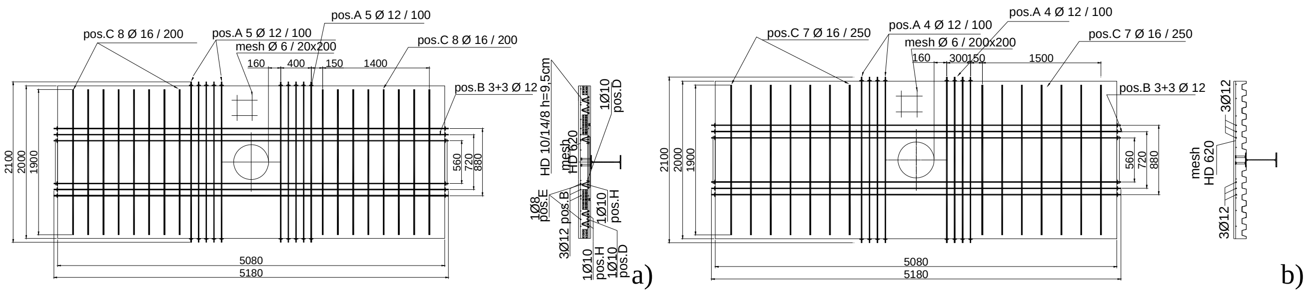


Figure 2 Slab reinforcement of a) a prefabricated lattice girder slab, b) a profiled steel sheeting slab

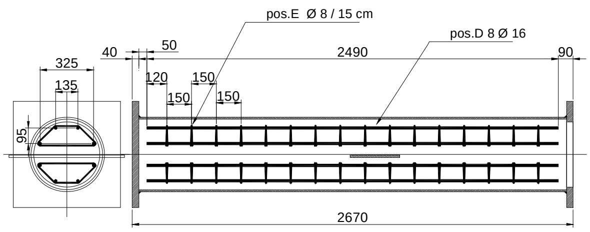


Figure 3 - Column stub and reinforcements capable of hosting a through-column web plate

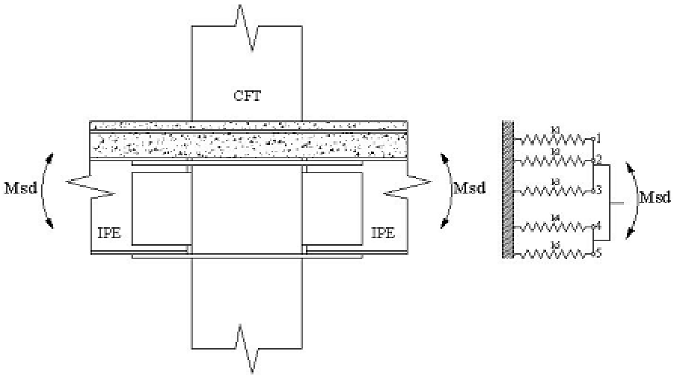


Figure 4 Mechanical model of a steel-concrete composite interior joint including sagging moments

3. TEST PROGRAM ON JOINTS

The experimental program regarded the execution of ten seismic tests and six fire tests on full-scale substructures representing interior and exterior welded steel-concrete composite beam-to-column joints with concrete filled tubes. Seismic tests were carried out at the University of Trento and at the University of Pisa, Italy, respectively, considering both cyclic and monotonic loadings. Conversely, fire tests were conducted at the Building Research Establishment, UK, with asymmetric loading on joints in order to simulate adjacent primary beams of different length. Additional information can be found in Bursi et al. (2008c).

3.1 Seismic tests

The experimental program concerned the execution of ten tests on full-scale substructures representing the interior and exterior welded steel-concrete composite beam-to-column joints with concrete filled tubes under seismic conditions. Six experimental tests on interior joints, i.e. 4 cyclic and 2 monotonic tests, were carried out at the University of Trento, Italy; while the remaining four experimental tests, on exterior joints, i.e. 3 monotonic and 1 cyclic test, were conduced at the University of Pisa, Italy.

3.2 Fire tests

The objective of this experimental program was to evaluate the fire resistance of joints following damage induced by an earthquake. In detail, fire tests were conduced on two undamaged and four pre-damaged specimens. The performance criterion was that joints should be capable of demonstrating 15 minutes fire resistance once damaged by earthquake effects without any additional fire protection. Therefore, four specimens were subjected to equivalent static load tests in order to simulate the effects of an earthquake as described in the next section.

4. TEST RESULTS ON JOINTS

As said before, in this paper only damage and fire tests results on joints are presented. Relevant seismic tests results are reported and commented in a companion paper (Bursi et al., 2008a).

4.1 Damage test results

In order to accurately simulate damage owing to a seismic event, non-linear dynamic time histories were performed by using the IDARC-2D program (Valles et al., 1996). Experimental data of joints were used to define both hysteretic laws in IDARC-2D and damage domains according to the Chai & Romstad criterion (Chai et al., 1995). Largest values of the damage index were not so high and about 0.43 for exterior joints and 0.34 for interior joints, respectively. Then, specific displacements were imposed before fire tests, in order to simulate seismic damage. The fire experimental tests are listed in Table 3.1. In detail, only one of each type of interior joints was subject to the pre-damage tests while all exterior joint specimens were pre-damaged.

Table 3.1 Type 2 joint specimens under fire loading

Test n.	Name	Concrete Slab Type	Maximum atmosphere temperature (°C)	Maximum steel temperature (°C)	Test duration (min)	Comments
F2-1	T21	Interior pre-damaged steel sheeting	1024	747	40	Test terminated due to runaway deflection - full depth cracking and seperation between steel sheet and slab.
F2-2	T22	Interior undamaged steel sheeting	970	966	60	No permanent deformation.
F2-3	T23	Exterior pre-damaged steel sheeting	972	963	60	No permanent deformation.
F2-4	T24	Interior pre-damaged precast lattice girder	1196	810	34	Test terminated due to runaway deflection - local buckling of the lower flange.
F2-5	T25	Interior undamaged precast lattice girder	982	721	45	Test terminated due to runaway deflection - cracking and spalling of concrete.
F2-6	T26	Exterior pre-damaged precast lattice girder	944	726	56	Test terminated due to runaway deflection - local buckling of the lower flange.

During damage tests the load, deflection and rotation of the slab were measured. The hydraulic actuators located in the basement of the lab were connected to the specimens through threaded bars connected to load cells. The actuators were displacement controlled in order to achieve the target deflection, load and rotation. Table 4.1 summarises the significant results from the damage tests. Moreover, both Figures 5 and 6 show the damage tests results in terms of load-deflection and moment-rotation relationships. Along the line of limited damage values, pre-damage tests resulted in little permanent deformation for the precast specimens; conversely, the applied load led to a separation between the steel sheeting and the concrete and both through depth and longitudinal cracks for the specimens with profiled steel sheeting.

Table 4.1 Summary of results for Type 2 joint under pre-damage loadings

Test n.	Specimen label	Max load (kN)	Max deflection (mm)	Max rotation (mrad)	Max Moment (kNm)
D2-1	T21	424	32	10	887
D2-2	T23	258	58	7.4	541
D2-3	T24	425	21	7.3	893
D2-4	T26	398	110	12.6	836

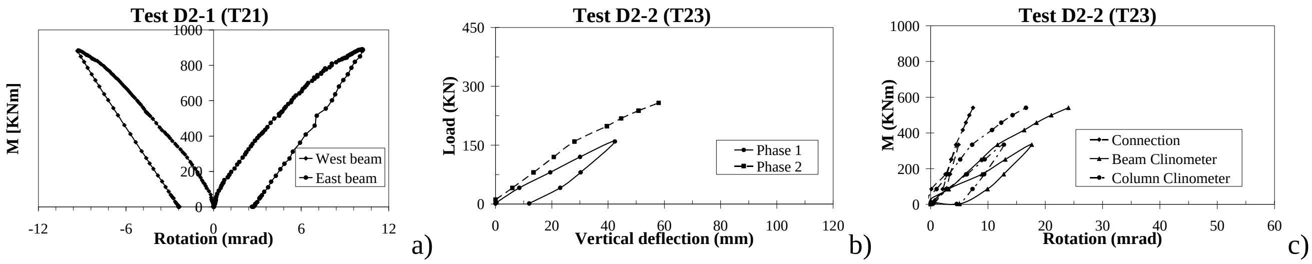


Figure 5 Specimen responses to pre-damage tests in the case of steel sheeting slabs

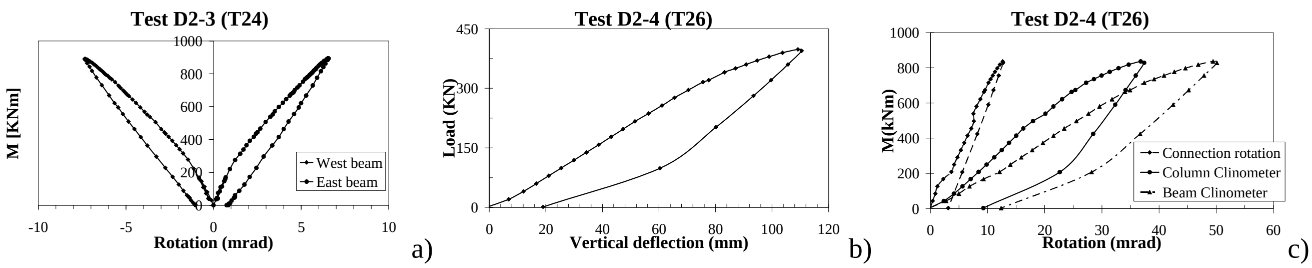


Figure 6 Specimen responses to pre-damage tests in the case of precast lattice girder slabs

4.2 Fire test results

The two interior precast specimens, pre-damaged and undamaged, lasted for one hour duration of fire. In detail, the behaviour of the precast specimen that was subjected to pre-damage testing was very similar to that of the undamaged specimen. Conversely, none of the specimens with profiled steel sheeting lasted for one hour duration of fire. In general failure occurred owing to an excessive rate of deflection at around 40 minutes. The test on the interior specimen subjected to damage testing terminated after approximately 34 minutes due to runaway deflection. As a result, the performance of the specimens in fire tests does not seem to be heavily influenced by the effects of the pre-damage test. In fact, Figure 7a shows a comparison between the behaviour of pre-damaged and undamaged composite specimens, whereas Figure 7b shows a comparison between the pre-damaged and undamaged precast interior specimens.

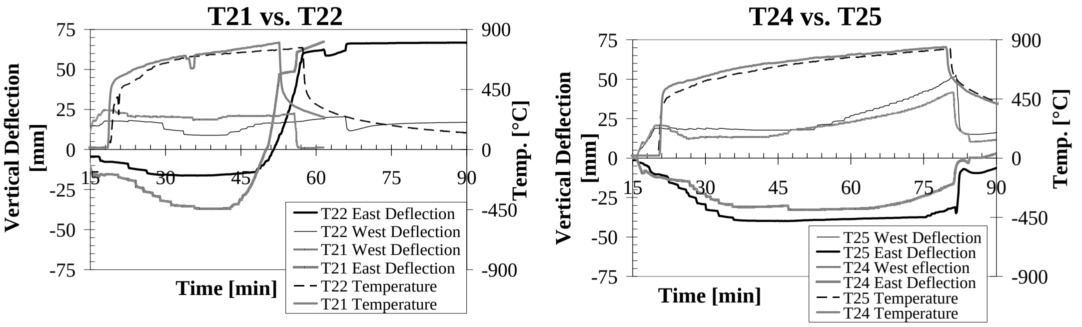


Figure 7 Fire test results: a) comparison between pre-damaged (T21) and undamaged (T22) steel sheeting specimens; b) comparison between pre-damaged (T24) and undamaged (T25) precast specimens

The graph illustrates an increased stiffness and strength associated with the precast slab that exhibits lower deformations for a given load and temperature.

5. SIMULATIONS ON FRAMES AND JOINTS

Initially and after fire tests on joints, structural analyses were performed on the whole frames to assess their global behaviour under the combined action of vertical, lateral and fire loadings. Moreover, on the basis of test data, FE models of joints were calibrated with the Abaqus 6.4.1 software (Hibbitt et al., 2000) and thermal analyses of joints were performed in order to obtain internal temperature distributions as a function of time.

5.1 Fire analysis of moment resisting frames

Numerical simulations on two-dimensional (2D) frames, were first performed by means of the SAFIR software (Franssen, 2000), in order to study different fire scenarios acting on the reference buildings; and to evaluate the performance of different elements, i.e. composite beams, composite columns, and beam-to-column joints, under the fire loading at different times of fire exposure. Five fire scenarios were considered. In the first one (FS1), fire acted only into a span in the first floor as illustrated in Figure 8a. In the second one (FS2), fire acted on the whole first floor; this means that the first level columns and the first level beams were heated. In the third one (FS3), fire acted only into a span in the last floor. In the fourth one (FS4), fire acted on the whole fifth floor. In the fifth one (FS5), fire acted on the whole frame. The fire load followed the ISO 834 (ISO 834-1, 1999).

a) FS1 b) FS2 c) FS3 d) FS4 e) FS5

Figure 8 Fire scenarios considered in thermal analyses

Figure 9 shows both the evolution of the bending moment and of the axial force as a function of time at various locations of the frame for the specific case FS1.

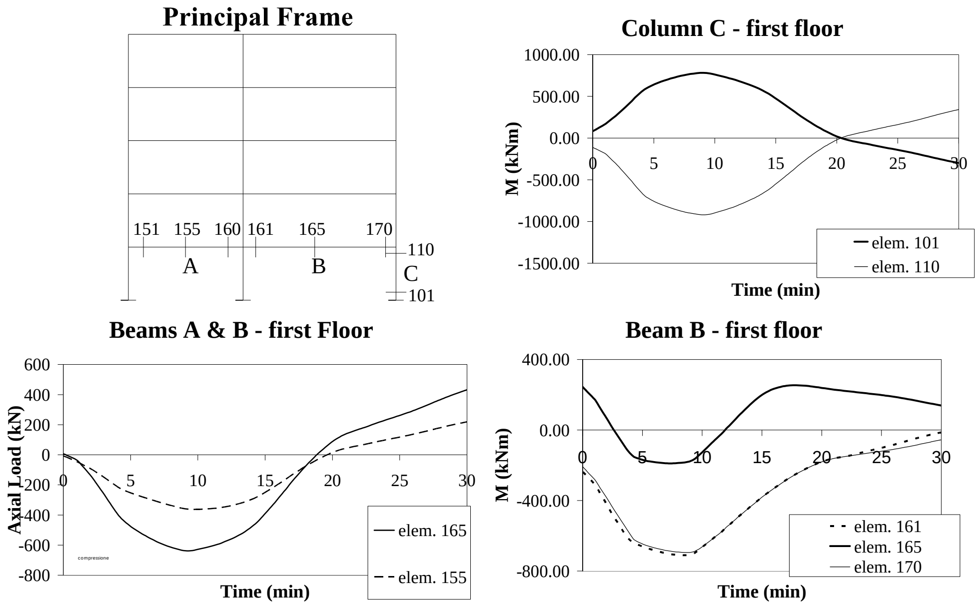


Figure 9 – Fire Case FS1: Bending moment and Axial load in beams and columns at the first storey

In detail, the bending moment in the column C shows a sign inversion when axial forces in beams change sign too. Initially indeed, the increase of temperature causes an increment of axial load in the beam -compression- up to about 18 min owing to the presence of the column restraint. Then, the reduction of stiffness of columns subject to fire prevails and the axial force in the beam turns out to tension, becoming similar to a *catenary* structure characterized by large deflections. The elongation of the beam owing to increase in temperature and the different *restraint* effects offered by the columns also cause a sign reversal of the bending moments at mid-span of the beam which from sagging becomes hogging and then sagging again. These results show clearly that the frame approaches collapse because of the formation of a beam mechanism in the longest span involving the formation of three plastic hinges located at mid-span and at both beam ends, respectively.

5.2 Fire analysis of joints

After calibration on fire test results, the Abaqus 6.4.1 software (Hibbitt et al. 2000) was employed to conduct FE thermal analysis of joints for different times of fire exposure, i.e. 15, 30 and 60 min, respectively. Moreover, it was employed to define some components at high temperature of the mechanical model sketched in Figure 4. For instance, Figure 10 shows the important reduction of the design moment capacity of the joint as a function of time for different fire exposures. In detail, the hogging capacity moment becomes approximately the 20 per cent of the initial value after 30 minutes of exposure; while it approaches about 5 per cent of its initial value after 60 minutes. Nevertheless, this time is enough to quit the building after a 0.4 g p.g.a. earthquake with a 10 per cent of probability of exceedence in 50 years.

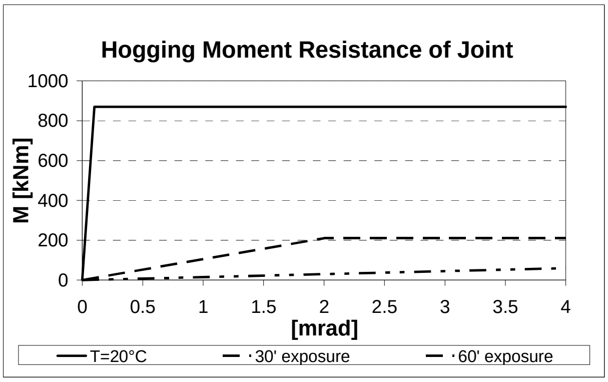
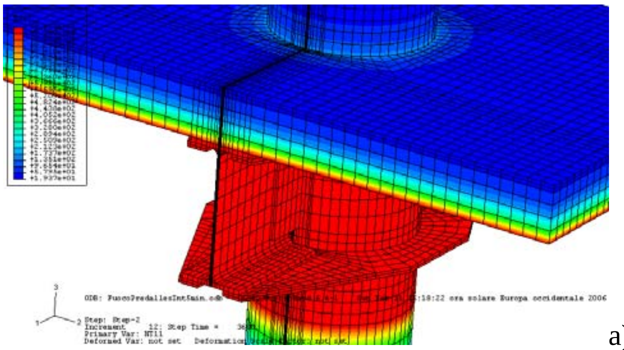


Figure 10. Moment-rotation relationship of a joint as a function of time at different fire exposures

In addition, FE models of joints shows clearly, e.g. Figure 11 at one hour of fire exposure, that joints endowed with prefabricated slabs exhibit a more favourable life behaviour compared to joints endowed with steel sheetings. In fact and for the latter, the steel portion of the column can directly heat the concrete slab near the joint and the steel sheet elements exhibit elevated temperatures even at short times of fire exposure.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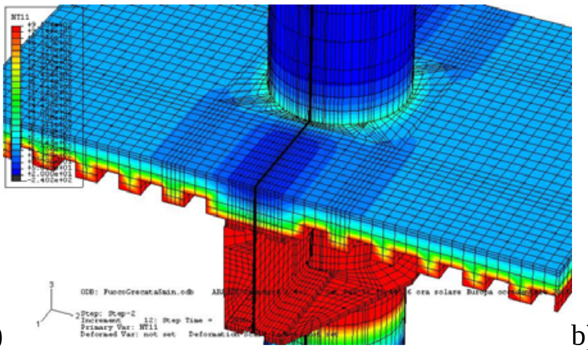
b)

Figure 11. Beam-to-column joint fire behaviour predicted by the Abaqus FE model: a) slab with a prefabricated lattice girder; b) slab with a steel sheeting

6. CONCLUSIONS

The paper has presented a multi-objective advanced design methodology dealing with seismic actions followed by fire on steel-concrete composite full strength beam-to-column joints with concrete filled tubes. In detail, fire experimental tests conduced on undamaged and pre-damaged joints were presented. Preliminary thermal analyses showed that joints endowed with prefabricated slab exhibit a more favourable behaviour compared to joints with composite slabs. This trend was confirmed by the results of experimental tests, where specimens with precast slabs exhibited lower deformations for a given load and temperature. Moreover, experimental results showed that: there was no noticeable difference in the fire performance between damaged and undamaged specimens both with precast and steel sheeting slabs owing to the inherent reliability involved in the seismic joint design with relevant Eurocodes; precast slabs performed better in fire tests than the corresponding specimens with steel sheeting at a fire exposure time in excess of the 15 minutes requirement; fire tests demonstrated the ability of full strength composite joints to concrete filled circular hollow sections to survive a damage equivalent to that corresponding to a design seismic event with a 0.4 g p.g.a. earthquake and a 10 per cent of probability of exceedence in 50 years. Finally, thermal analysis on two-dimensional (2D) frames were presented together with relevant refined thermal analyses of joints. The latter analyses allowed the component method of steel-concrete composite joints at high temperature to be calibrated and exploited for predictions.

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