Steel-concrete composite full strength joints with concrete filled tubes: design and test results

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ABSTRACT: In this paper, a multi-objective advanced design methodology dealing with seismic actions followed by fire on steel-concrete composite full strength joints with concrete filled tubes is proposed instead of a traditional single-objective design where fire safety and seismic safety are achieved independently. Experimental tests together with numerical simulations of the fire behaviour were carried out to derive fundamental information about the performance of these joints. In detail a total of six specimens designed according to Eurocode 3, 4 and Eurocode 8 were subjected to monotonic and cyclic loadings up to collapse at the Laboratory for Materials and Structures Testing of the University of Trento, Italy. These specimens were detailed in order to exhibit a favourable fire behaviour after a severe earthquake. The major aspects of the cyclic behaviour of composite joints are presented and commented upon together with fire analysis results. Both the experimental activity and the numerical FE simulations demonstrate the adequacy of the proposed joint design.

1 INTRODUCTION

In the design of steel-concrete composite buildings the sequence of seismic and fire loadings are not taken into account. In fact, the seismic safety and the fire safety are considered separately. In reality, the risk of loss of lives increases if a fire occurs within the building after an earthquake. Where significant earthquakes can occur, fire after earthquake is a design scenario that should be properly addressed in any performance-based design.

This approach takes into account seismic safety and fire safety with regard to accidental actions as well as fire safety on a structure characterized by stiffness deterioration and strength degradation owing to seismic actions. The proposed design solution, were developed in an European research (Colombo & Bursi 2006). This research program is intended to develop fundamental data, design procedures and promotion of two types of ductile and fire-resistant composite beam-to-column joints with:
1. partially reinforced-concrete-encased column with I-section;
2. concrete filled tubular column with circular hollow steel section.

The project analyses the scenario in which a fire follows and earthquake, thus defining joint typologies for which, after being damaged by an earthquake, a residual load-bearing capacity is assured during a fire occurring after an earthquake. The design is performed in the modern context of performance-based engineering determining both the stiffness deterioration and the strength degradation of composite joints after seismic loading.

The present paper presents the results of experimental tests under monotonic and cyclic loading together with numerical simulations of the fire behaviour of steel-concrete composite full strength joints with concrete filled tubes.

2 COMPOSITE BUILDING DESIGN

The actions used in the design of the proposed joints were obtained by the analyses of two moment resisting frames having the same structural typology but different slab systems (160 mm composite steel-concrete slab high with structural profiled steel sheeting and 160 mm concrete slab high composed of electro-welded lattice girders).

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 that takes into account the different load bearing capacities of the two slab systems as well as the need to avoid propping systems during the construction phase. As a result, the main moment resisting frame is made up by two bays spanning 7.5 m and 10.0 m
in the solution with steel sheeting with a distance between secondary beams equal to 2.5 m; and by two bays spanning 7.0 m and 10.5 m in the solution with lattice steel girders with a distance between secondary beams equal to 3.5 m (Figs 1, 2). All slabs were arranged in the direction parallel to the main frames. The main beams were IPE 400 while the secondary ones were IPE 300.

2.1 Composite Beams

Two different types of composite beams were checked according to point 6.1.1 of Eurocode 4 (UNI EN 1994-1-1. 2005). In the first one, the beam section was an IPE400 with steel grade S355 while the deck was a composite slab with a prefabricated lattice girder made by the Pittini Group (Fig. 3).

![Figure 1. Plan and structure typology; slabs with prefabricate lattice girders. Dimensions in metres.](image1)

![Figure 2. Plan and structure typology, slabs with profiled steel sheetings. Dimensions in metres.](image2)

![Figure 3. Specimens WJ-P - Slab with prefabricate lattice girders. Dimensions in mm.](image3)

![Figure 4. Specimens WJ-S - Slab with profiled Steel Sheetings. Dimensions in mm.](image4)

2.2 Composite Columns

The columns of steel grade S355 were concrete-filled column with a CFT filled tubular column steel profile with a diameter of 457 mm and a thickness of 12 mm. Column reinforcements consisted of 8φ16 longitudinal steel bars and stirrups φ8@150 mm.

The concrete class of composite columns was C30/37, while the steel grade S450 was adopted for the reinforcing steel bars as illustrated in Figure 5.
2.3 Beam-to-Column Composite Joints

The joint consisting of a steel concrete composite beam attached to a concrete-filled tubular column with circular hollow steel sections and is composed by two horizontal diaphragm plates and a vertical through-column plate (Figures 6 and 7). The presence of a vertical through column plate shown in Figure 7 allows the transmission of shear forces owing to vertical loads from a beam to the other.

The following components were considered in the method: concrete slab in compression; upper horizontal plate in compression; vertical plate in bending and lower horizontal plate in tension, for sagging moment; reinforcing bars in tension, upper horizontal plate in tension; vertical plate in bending and lower horizontal plate in compression for hogging moment. The components concrete slab in compression and upper horizontal plate in tension were identified by means of FE models set with ABAQUS (Hibbitt et al., 2000). Figure 9 shows the FE model of a plate in tension to identify the effective width b.

![Figure 9. FE model of a plate in tension: (a) elastic stresses; (b) inelastic stresses and width.](image)

3 EXPERIMENTAL TESTS

A total of 6 specimens was designed and fabricated according to Eurocode 4 (UNI EN 1994-1-1. 2005, UNI EN 1994-1-2. 2005) and Eurocode 8 (UNI EN 1998-1. 2005) provisions. The test specimens were interior subassemblage and the joints were made by two horizontal diaphragm plates split into two equal halves along the diagonal for fabrication convenience and easiness of assembling.

Once each side is properly placed on the pipe the two halves are attached with a full-joint penetration groove weld. In detail, the inferior plates are welded to column in the shop, while the superior plates are welded on site as should be understood from Figure 6. Flanges and web of each beam were connected to the horizontal plates and the vertical plate by welding, as depicted in Figure 7.

In all composite specimens the connections between steel beam and desk were made by Nelson 19 mm stud connectors with an ultimate tensile strength $f_u=450$ MPa. The joints differ each other owing to the slab type and to additional Nelson 19 mm studs localized around the column in order to enforce a better force transmission between the column and the composite slabs as indicated in Figure 7. The joint specimens were subjected to monotonic and cyclic loadings up to collapse, according to the ECCS stepwise increasing amplitude loading protocol, modified with the SAC procedure (ECCS 1986,
SAC 1997): $e_y=17.5$ mm. Hereafter the specimens are indicated as follows:
- WJ-P1 – specimens with electro-welded lattice girders slab and no Nelson connectors around the column;
- WJ-P2 – specimens with electro-welded lattice girders slab and Nelson connectors around the column;
- WJ-PM – specimens with electro-welded lattice girders slab and no Nelson connectors around the column;
- WJ-S1 – specimens with profiled Steel Sheeting slab and no Nelson connectors around the column;
- WJ-S2 – specimens with profiled Steel Sheeting slab and Nelson connectors around the column;
- WJ-SM – specimens with profiled Steel Sheeting slab and no Nelson connectors around the column;

3.1 Test Set-Up

The test set-up was designed in order to simulate the conditions of interior beam-to-column joints with concrete filled tubes within frame structures.

It consisted of a reaction wall, a hydraulic actuator (capacity 1000 kN, stroke ±250 mm), a reinforced-concrete slab, a lateral frame designed to prevent specimen lateral displacements.

The main instrumentation employed is indicated in Figure 10 and detailed herein:
- 5 inclinometers were utilized in order to measure the inclinations of the zone adjacent to the joint and of the beams near the connection;
- 4 LVDTs detected the interface slip between the steel beam and the concrete slab and between the inferior horizontal plates and the beam flange;
- 2 LVDTs were employed in order to measure the connection deformation;
- 10 LVDTs were utilized in order to measure concrete slab deformations;
- 4 Omega strain gauges detected the deformations of the concrete slab;
- 8 strain gauges monitored axial deformations of the reinforcing bars in order to scrutinise the effective breadth of the reinforcing bars at each loading stage;
- 4 strain gauges monitored deformations of superior and inferior horizontal plates;
- 4 strain gauges recorded flange strains in order to estimate internal forces in steel beams;
- 2 load cell were set on the top of pendula and were utilized in order to measure horizontal and vertical components of axial forces;
- 1 digital transducer (DT500) was employed in order to measure the top column displacement;
- a FieldPoint acquisition system in order to acquire experimental data.

Figure 10. Main instrumentation on specimens.

3.2 Results

Experimental results are presented on the basis of hysteretic responses. In particular, Force-displacement (F-d) curves for all tested specimens are shown in Figures 11-14, while moment vs. rotation (M-φ) relationships are depicted in Figures 15-18.

All specimens exhibit a good performance in terms of resistance, stiffness, energy dissipation and ductility.

Both the overall force-displacement relationships and the moment-rotation relationships relevant to plastic hinges formed in the composite beams exhibit a hysteretic behaviour with large energy dissipation without evident loss of resistance and stiffness. In particular we can be observe that the hysteretic loops of moment-rotation relationship are unsymmetrical due to the different flexural resistance of the composite beam under hogging and sagging moments, respectively.

For all specimens, the experimental tests show a remarkable and progressive deterioration of strength, stiffness and energy absorption capacity as a consequence of the formation of a plastic hinge associated with local buckling of the beam flange. The collapse of all specimens was associated with cracking of beam flanges.
Figure 13. Specimen WJ-PM- Force-displacement relationship.

Figure 14. Specimen WJ-SM- Force-displacement relationship.

Figure 15. Specimens WJ-P1 and WJ-P2 - Moment-rotation relationships of plastic hinges.

Table 1 reports some major experimental results for each Test: in detail, the maximum applied Displacement ($d$), the maximum value of the Force ($F$), maximum values of both Sagging and Hogging Moment ($M$) and corresponding values of initial stiffness ($k_0$), total number of Cycles performed during the tests ($N_{tot}$) are collected.

Monotonic tests were interrupted before failure of the specimens in order to reduce the risk of damaging the test equipment.
friction existing between the concrete slab and the composite column. Therefore, in order to activate some of the transfer mechanisms proposed in Eurocode 8, (i.e. Mechanism 1 and Mechanism 2), it was necessary to increase the level of friction between the concrete slab and the composite column of specimens.

As a result, Nelson stud connectors welded around the column were adopted in some specimens along the details indicated in Figure 7. Experimental results showed that the activation of the aforementioned mechanisms was evident and effective in the specimens fabricated with Nelson connectors welded around the column; while in the specimens fabricated without stud connectors around the columns Mechanisms 1 and 2 were less effective.

Table 1: major experimental data

<table>
<thead>
<tr>
<th>Specimen</th>
<th>d (mm)</th>
<th>E (kN)</th>
<th>M (kNm)</th>
<th>kN/mrad</th>
<th>Ntot</th>
</tr>
</thead>
<tbody>
<tr>
<td>WJ-P1</td>
<td>210</td>
<td>+714.80</td>
<td>+962.46</td>
<td>+181.27</td>
<td>21</td>
</tr>
<tr>
<td>WJ-P2</td>
<td>210</td>
<td>-686.30</td>
<td>-767.72</td>
<td>-141.00</td>
<td>21</td>
</tr>
<tr>
<td>WJ-S1</td>
<td>210</td>
<td>+708.90</td>
<td>+939.07</td>
<td>+183.43</td>
<td>21</td>
</tr>
<tr>
<td>WJ-S2</td>
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<td>+695.40</td>
<td>+768.48</td>
<td>-158.00</td>
<td>21</td>
</tr>
<tr>
<td>WJ-PM</td>
<td>490</td>
<td>+661.70</td>
<td>+296.51</td>
<td>+220.32</td>
<td>Mon.</td>
</tr>
<tr>
<td>WJ-SM</td>
<td>490</td>
<td>+662.00</td>
<td>-693.77</td>
<td>-113.50</td>
<td>Mon.</td>
</tr>
</tbody>
</table>

Figure 20 shows the total dissipated energy i.e. the Cumulative Energy for the specimens belonging to the WJ-P and WJ-S series. The dissipated energy in the plastic cycles is practically identical for all the specimens; after the 15th cycle which corresponds to a displacement equal to 10\(\varepsilon_y\), specimens with electro-welded lattice girders slab dissipate more energy than those with profiled steel sheetings owing to a better influence of the force transfer between columns and slabs on plastic hinges.

Moreover, four fire tests on full-scale substructures were performed too. In particular, two specimens, labelled as T21 and T24, were pre-damaged to simulated damage owing to Type 1 spectrum compatible accelerograms at 0.4g pga (UNI EN Eurocode 8-1, 2005), and two specimens (T22 and T25)
were not, to clearly appreciate seismic damage effects on fire resistance. The Temperature vs. time curve imposed to the specimens T21-T22 and T24-T25 is shown in Figure 21(a) and (b), respectively. Moreover, thermal analyses with different fire scenarios were carried out by means of the SAFIR code (Franssen 2000) on a two-dimensional frame model. Figure 22 shows the considerable reduction of the design capacity moment of the joint as a function of the time of fire exposure.

Figure 22. Moment-rotation relationships of the joint as a function of the time of fire exposure.

Specimens T21 and T22 with profiled steel sheeting slabs exhibited failure owing to an excessive rate of deflection at approximately 40 minutes. The test on specimen T21 terminated after approximately 34 minutes owing to runaway deflection. Following the fire test, the profiled steel sheeting separated from the slab; then the slab cracked both along the surface and through the depth with extensive buckling at one hour both of the lower flange and the web of the adjacent east beam.

T24 and T25 specimens endowed with prefabricated slabs endured one hour of fire; however, in both cases specimens were very close to failure as indicated in Figure 21b, by an increasing rate of deflections towards the end of the test. However, at this stage, there was no permanent deformation and no sign of any significant damage from fire tests.

4 NUMERICAL SIMULATIONS

Different fire scenarios acting in the reference building were studied with the objective to evaluate the performance of composite beams, composite columns and beam-to-column joints under fire load for different times of exposure.

The Abaqus code (Hibbitt et al. 2000) was employed to perform thermal analyses of joints for different times of fire exposure, i.e. 15 min, 30 min and 60 min, respectively.

Moreover, thermal analysis conducted by means of the Abaqus 6.4.1 code, for different times of fire exposure, demonstrated that the joint endowed with prefabricated slab exhibits a better fire behaviour compared to the joint endowed with steel sheeting as illustrated in Figure 24. Anew, this is due to the uniform thickness of the lattice girder with respect to the profiled steel sheeting.
In sum we can affirm that both experimental and numerical results demonstrated the adequacy of the proposed joint design for concrete filled tubes to adequately face earthquake loading followed by fire.

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REFERENCES


5 CONCLUSIONS

In this paper, a multi-objective advanced design methodology was proposed instead of a traditional single-objective design. Experimental cyclic tests and numerical simulations regarding the fire behaviour of joints were carried out. Experimental results show how joint details influence beam-column subassemblage responses both in terms of seismic and fire performance.

In detail, test results exhibited rigid behaviour for the designed composite joints; as expected plastic hinges developed in beams owing to the capacity design.

It was evident that owing to local buckling effects, severe strength degradations appeared to exceed 20 per cent of maximum strength values with rotations of about 40 mrad. As a result, these joints are not suitable for high ductile structures to be used under seismic loading according to Eurocode 8 (UNI EN 1998-1. 2005). Nonetheless, the corresponding minimum plastic limit of 35 mrad remains rather high for typical European earthquakes.

Thermal analyses showed that steel elements, directly exposed to fire loading, had the same temperature of air, while for the concrete slab different behaviours were observed depending on the adopted typology. In fact, specimens with profiled steel sheetings slab were characterized by temperatures higher than those endowed with electro-welded lattice girders. Moreover fire experiments showed the favourable behaviour of joints.