SEISMIC PERFORMANCE OF WELDED STEEL-CONCRETE COMPOSITE BEAM-TO-COLUMN JOINTS WITH CONCRETE FILLED TUBES

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ABSTRACT
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 presented in this paper. Instead of a traditional single-objective design where fire safety and seismic safety are achieved independently and the sequence of seismic and fire loadings are not taken into account, the new design approach should guarantee: i) seismic safety with regard to accidental actions; ii) fire safety with regard to accidental actions; iii) fire safety on a structure characterized by stiffness deterioration and strength degradation owing to seismic actions.
In the seismic design of steel-concrete composite joints the component method was utilized, while the criteria of Eurocode 4 were exploited for fire design. In detail, these specimens were detailed in order to exhibit a favourable fire behaviour after a severe earthquake and designed according to Eurocode 3, 4 and Eurocode 8.
The research activity was mainly concerned with the combined design of the proposed joints with concrete filled tubes together with the definition of some structural details for composite beams, such as the layout of structural steel reinforcements into the slab; and the definition of some details for the stud connectors around the column close the steel beam-to-column connection.
In this paper the results of the experimental program on joints are described and the experimental results are discussed together with numerical simulation results on frames. The experimental activity demonstrated the adequacy of both the seismic design and performance of beam-to-column joints.

KEYWORDS
Seismic design, steel-concrete composite joint, fire design, fire safety, experimental results, numerical simulations.

1. INTRODUCTION

In the design of steel-concrete composite buildings in Europe the seismic safety and the fire safety are considered separately and the sequence of seismic and fire loadings are not taken into account. Indeed, the risk of loss of lives increases if a fire occurs within the building after an earthquake. In this respect, seismic-induced fire is a scenario with high probability of occurrence as highlighted by recent earthquakes in Northridge 1994 and Kobe 1995. For instance following the Kobe earthquake, Sekizawa et al. (2003) showed clearly through computations, see Figure 1, that the probability of exceeding more and more severe fire phases becomes larger as p.g.a. increases. Moreover, studies of future scenario large-scale earthquakes in San Francisco and Tokyo indicated that seismic-induced fire is an important factor in the subsequent damage to property and loss of lives (Wellington Lifelines Group, 2002). It is obvious therefore, that fire after earthquake is a design scenario that should be properly addressed in any performance-based design, especially in locations where significant

earthquakes can occur. Earthquakes then, increase the risk of loss of life if a fire occurs within a building.
A new approach to this problem is proposed in this research: to pursue a multi-objective design criterion where fire safety and seismic safety are simultaneously achieved. This approach takes into account seismic safety and

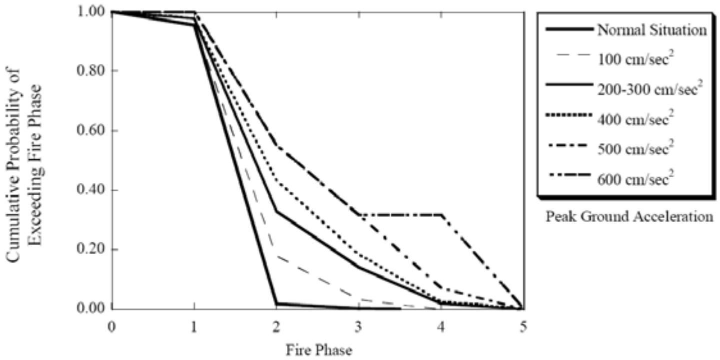


Figure 1 Probability of exceeding fire phases vs. p.g.a. (cm/sec^2) after Sekizawa et al. (2003).

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. As a result, the fire design applied to a structure with reduced capacity owing to seismic actions will achieve structural, seismic and fire safety as required; but also structural and fire safety and structural and seismic safety will be guaranteed, respectively. The present research activity was part of an European research project (PRECIOUS), 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., 2008):

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.

The project achieved its goals through a balanced combination of analytical/numerical and experimental work on seismic and fire actions. This paper focuses on seismic experimental program results relevant to Type 2 joints together with numerical simulation results on moment resisting frames.

2. DESIGN OF REFERENCE FRAMES UNDER SEISMIC AND FIRE LOADINGS

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. The composite steel-concrete office building was endowed with 5 floors with 3.5 m storey height as shown in Figure 2. 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 systems during the construction phase. All slabs were arranged in parallel to main frames.
Two different types of slab were checked employed. In the first one, deck was a composite slab with a prefabricated lattice girder, see Figure 3a, with slab reinforcement performed by 3+3ϕ12 longitudinal steel bars and by 5+5ϕ12@100 mm plus 8+8ϕ16@200 mm transversal steel bars. A mesh ϕ6@200x200 mm completes the slab reinforcement. In the second type of slab, observe Figure 3b, a composite slab with profiled steel sheeting was made with the same slab reinforcement. 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 fu=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 8ϕ16 longitudinal steel bars and stirrups ϕ8@150 mm.
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 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).
The seismic performance of frames was evaluated by means of nonlinear static analysis. Fire design was

considered and the structural fire performance of the complete frames was evaluated by means of the SAFIR program (Franssen, 2002) for different fire scenarios.

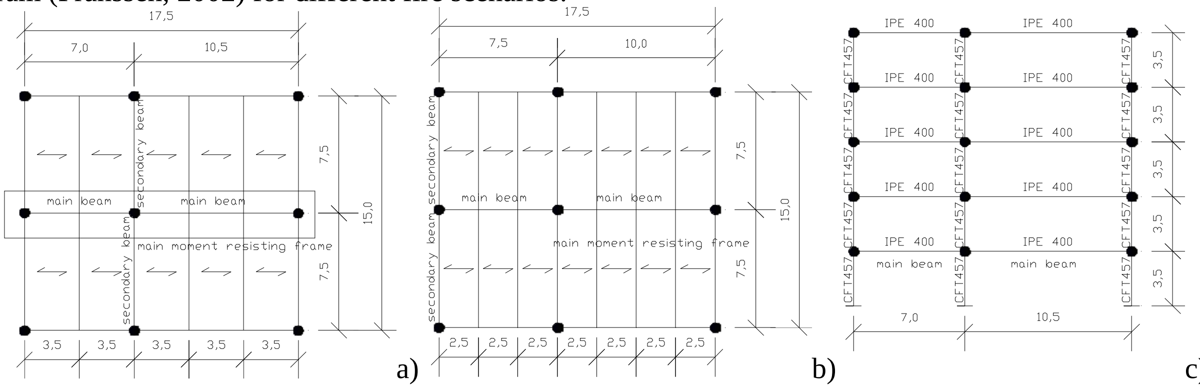


Figure 2 Geometric layout of reference structures: a) structure with slabs with prefabricated lattice girders; b) structure with slabs with profiled steel sheeting; c) frame elevation.

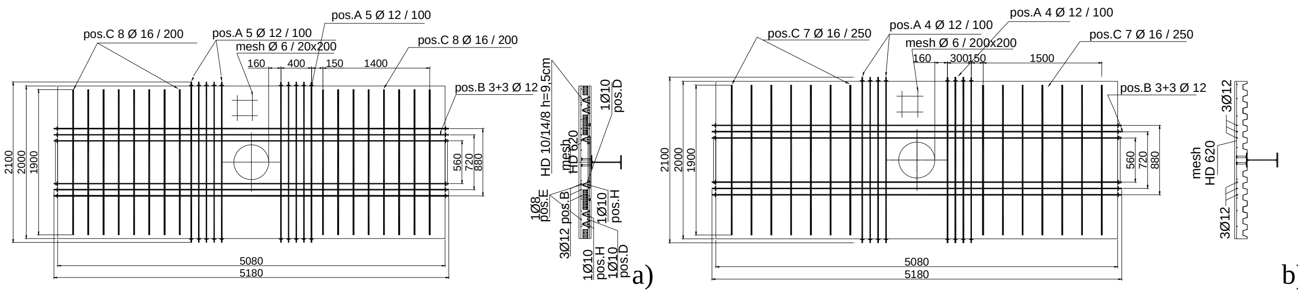


Figure 3 Slab reinforcements of a) a prefabricated lattice girder slab, b) a profiled steel sheeting slab.

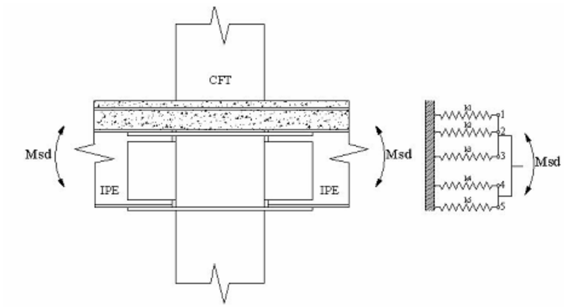


Figure 4 Mechanical model of an interior joint.

3. TEST PROGRAM

The experimental programme regarded the execution of ten seismic tests and six fire tests on full-scale substructures representing interior and exterior welded beam-to-column joints. Seismic tests were carried out at the University of Trento and at the University of Pisa, respectively, considering both cyclic and monotonic loadings. Pre-damage and fire tests were conducted at the Building Research Establishment, UK, with asymmetric loading on joints to simulate adjacent primary beams of different length.

3.1 Seismic tests

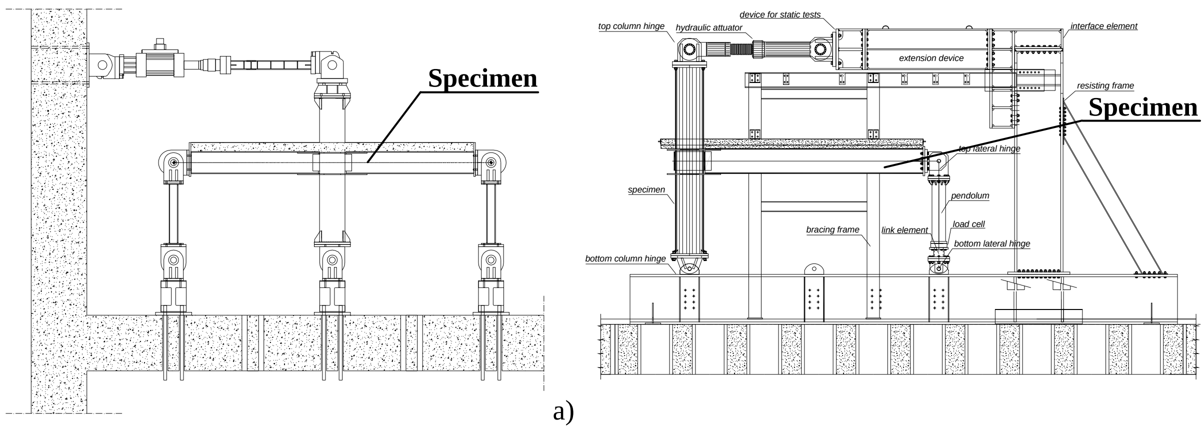
The experimental programme comprised ten tests on full-scale substructures that are listed in Table 3.1.
They represent both interior and exterior welded beam-to-column joints with concrete filled tubes. Six experimental tests on interior joints were carried out at the University of Trento while the remaining four experimental tests on exterior joints, were conduced at the University of Pisa. The relevant set-ups are shown in Figure 5. Specimens were subjected to monotonic and cyclic loadings up to collapse, according to the ECCS stepwise increasing amplitude loading protocol (ECCS, 1986), modified with the SAC procedure (Karl et al.,

1997). The test programme was conceived in a way to analyse different design solutions. Joints differ each other owing to the slab type and to additional Nelson 19 mm studs localized around the column to enforce a better force transmission between column and composite slabs. No axial load was applied to the column during monotonic and cyclic tests. Addition information on the experimental set-up and instrumentations can be found in (Bursi et al., 2008)

Table 3.1 Experimental program relevant to Type 2 composite joints

Test n.	Name	Configuration	Test Method	Concrete Slab Type
S2-1	WJ-PM	Interior joint: Solution 1 - no stud connectors	Monotonic	Prefabr. lattice girder
S2-2	WJ-SM	Interior joint:Solution 1 - no stud connectors	Monotonic	Steel sheeting
S2-3	WJ-P1	Interior joint: Solution 1 - no stud connectors	Cyclic	Prefabr. lattice girder
S2-4	WJ-P2	Interior joint: Solution 2 – with stud connectors	Cyclic	Prefabr. lattice girder
S2-5	WJ-S1	Interior joint: Solution 1 - no stud connectors	Cyclic	Steel sheeting
S2-6	WJ-S2	Interior joint: Solution 2 – with stud connectors	Cyclic	Steel sheeting
S2-7	S2-7	Exterior joint: Solution 1 - no stud connectors	Monotonic: Hogging moment	No slab
S2-8	S2-8	Exterior joint: Solution 1 - no stud connectors	Monotonic: Hogging moment	Steel sheeting
S2-9	S2-9	Exterior joint: Solution 1 - no stud connectors	Monotonic: Hogging moment	Prefabr. lattice girder
S2-10	S2-10	Exterior joint: Solution 1 - no stud connectors	Cyclic	Prefabr. lattice girder

Main results are presented and commented hereafter.



a) b)

Figure 5 Lateral view of the test set-up at a) the University of Trento; b) the University of Pisa

3.2 Fire tests

The objective of the experimental programme was to evaluate the seismic-induced fire resistance of joints. Fire tests were conduced on two undamaged and four pre-damaged specimens. The target performance criterion is that joints should be capable of demonstrating 15 minutes fire resistance once damaged by the effects of an earthquake without fire protections. Therefore four specimens were subjected to static load test to simulate the effects of an earthquake. In order to accurately simulate the damage owing to a seismic event, a specified displacement was imposed on the specimens before fire testing based on cyclic test results carried out.

4. TEST RESULTS ON JOINTS

In this paper only the seismic tests results, carried out in Trento and Pisa, respectively, are presented. Fire tests results are detailed in a companion paper presented at this conference (Alderighi et al., 2008).

4.1 Seismic test results of beam-to-column joints

4.1.1 Interior Joints

The force-displacement relationships of WJ-P1 and WJ-P2 specimens with electro-welded lattice slabs and without/with Nelson connectors around the column are illustrated in Figure 6a and 6b, respectively. Plastic hinges developed in beams adjacent to joint and progressive deterioration of strength and stiffness was associated with beam flange buckling. Failure was due to beam flange cracking. Not so much difference was noted by comparing the global behaviour of the two specimens; this implies that the presence of the Nelson connectors around the column does not influence the ductility of the joint, being the plastic mechanism localised in composite beams. Similar results were obtained for specimens WJ-S1, see Figure 6c, and WJ-S2, see Figure 6d, endowed with slabs with profiled steel sheeting and without/with Nelson connectors around the column, respectively. Differently from the two previous cases, i.e. the specimens endowed with prefabricated concrete slab, in both beam and weak section of the joint, the neutral axis was located on the beam web for sagging moment. This happens because the steel sheeting slab was more damaged compared to the prefabricated lattice slab. The overall force-displacement relationships relevant to plastic hinges formed in the composite beams exhibited a hysteretic behaviour with large energy dissipation without evident loss of resistance and stiffness. Nonetheless at the collapse onset, the experimental tests showed 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 the bottom beam flange and web.

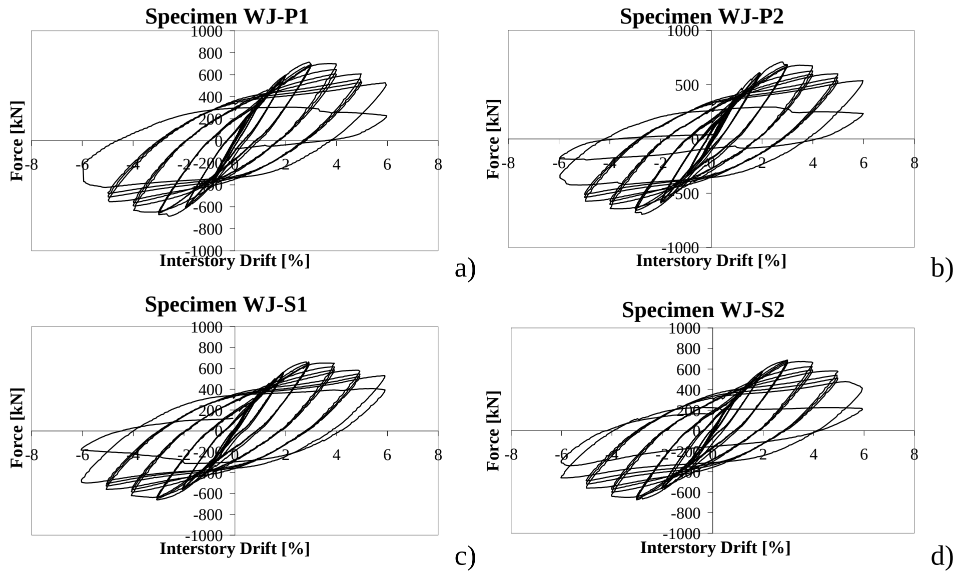


Figure 6 Results of cyclic tests on interior joints. Force vs. interstorey drift relationship of a), b) a prefabricated lattice girder slab; c), d) a steel sheeting slab

Monotonic test results for the specimens WJ-PM and WJ-SM without Nelson connectors welded around the columns are plotted in Figure 7a and 7b, respectively. Both specimens exhibited less damage owing to the lack of cyclic loading. Moreover, a favourable behaviour in term of strength of the specimen WJ-PM endowed with the lattice girders slab was evident owing to a better composite action in the section relevant to the plastic hinge.

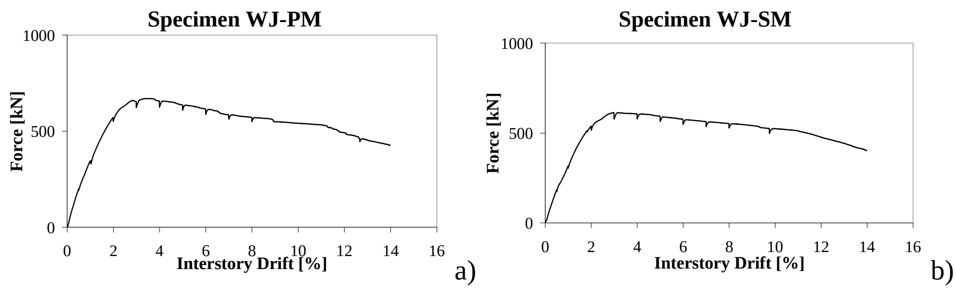


Figure 7 Monotonic test results of interior joints. Force vs. interstorey drift relationship of a) a prefabricated lattice girder slab; b) a steel sheeting slab

4.1.2 Exterior Joints

The bare steel solution, i.e. the specimen S2-7, revealed a good performance in terms of strength and ductility; as expected by design, the observed strength value was higher than that of the adjacent beam as illustrated in Figure 8a. Conversely, the solution including the steel sheeting deck, i.e. the specimen S2-8, revealed a good performance in terms of strength and ductility, without suffering the brittle failure of the concrete slab and reaching the maximum value of the interstorey drift as illustrated again in Figure 8a. Even in this case before collapse, a starting of buckling of the lower compressed flange was observed together with a web folding.

The solution with prefabricated lattice girders, i.e. the specimen S2-9, presented some problems in the rear column; in detail, an early crushing of concrete in that side was observed, thus preventing the obtainment of adequate values of strength and ductility. This was due to a lack of adequate connection between the two prefabricated parts of the concrete slab, thus causing an expulsion of the included concrete. Besides, the undesirable presence of a polystyrene panel in the back side represented the trigger point for concrete cracking. In this case even though only 4.5 per cent of interstorey drift was reached, the buckling in the compressed lower flange was observed.

The cyclic test S2-10 was performed on a specimen with a prefabricated lattice girder. This test was repeated twice, since the first time it was stopped due to failure of the welding connecting the horizontal plate to the inferior beam flange owing to imperfections. In this case, the damage of the composite slab consisted in a limited crushing of concrete in the front column and once again in an extensive crushing in the rear side owing to: i) the limited length of concrete in the exterior part of the slab; ii) the absence of suitable connecting devices between the two halves of the prefabricated lattice girder; iii) the undesirable presence of polystyrene panels in the rear side representing a trigger point for concrete cracking. The response representing the applied force vs. the top column interstorey drift after welding repairing is shown in Figure 8b.

The test was stopped because of tearing of the beam web owing to a sudden failure of the bottom steel flange. The outcome of these tests was the satisfactory behaviour of the solution including the corrugated steel sheeting; conversely, the solution with prefabricated lattice girders requires additional studies to improve structural details of the exterior side to avoid an early concrete crushing.

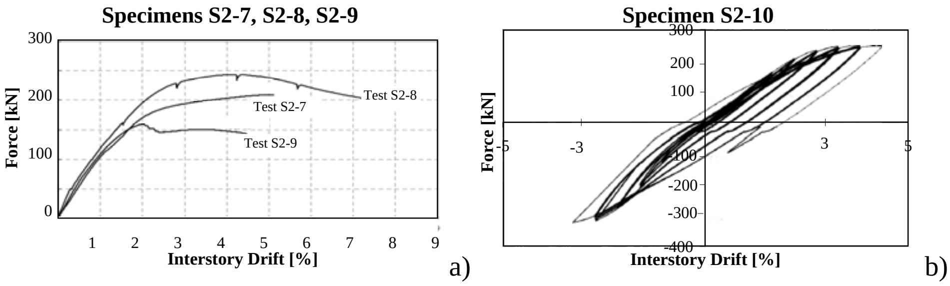


Figure 8 Force vs. interstorey drift relationships: a) Monotonic test results; b) Cyclic test results

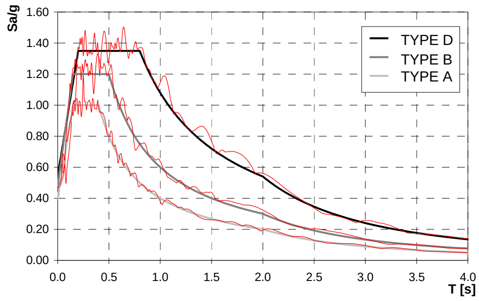
5. RESULTS ON FRAMES

Experimental tests carried out on interior and exterior joints showed that joints exhibited enough stiffness and strength to be considered as rigid and full strength according to Eurocode 4 and 8 (EN 1994-1-1, 2004, EN 1998-1, 2005). Nevertheless, two sets of simulations were performed at the frame level to assess both the seismic behaviour of moment resisting frames endowed with the proposed joints both with steel sheeting and prefabricated slabs (Bursi et al., 2008). The first set of simulations was performed by using pushover techniques through the software Ruaumoko (Carr, 2005). Two frames of five-storey and two-span, see Figure 2, were analysed. The frames were designed against seismic actions considering a Type 1 spectrum, with subsoil Class B (EN 1998-1, 2005), and a p.g.a. of 0.40g. All elements were defined as component members with plastic properties (Carr A.J., 2005). A tri-linear degrading hysteresis rule after Saiidi and Sozen (1979) was exploited in view of the definition of the dynamic behaviour of elements. Also the columns were defined by means of a strength degradation law based on the maximum developed ductility. The joint behaviour was modelled by using different rigid and spring elements based on the geometric properties of simulated joints (Bursi et al., 2008). Two different lateral load distributions were employed. The first one was a uniform lateral load, while the second one was proportional to the first modal shape. In both cases vertical loads and imperfections were included thus reducing both the initial stiffness and the natural frequencies of frames. These simulations were used to obtain capacity curves thus ductility factors and overstrength could be computed as listed in Table 5.1. Because the main mechanism of energy dissipation of the analysed frames was through plastic hinges at beam ends, pushover analyses entailed a rather favourable behaviour of these joint/frame typologies.

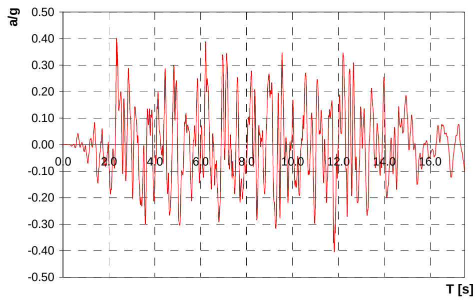
Table 5.1. Behaviour and overstrength factor estimates from pushover analyses.

	Prefabricated Slab		Steel Sheeting Slab	
	Ductility μ_y	Overstrength Ω	Ductility μ_y	Overstrength Ω
Uniform lateral load	6.10	2.10	10.06	3.25
Modal lateral load	6.10	2.21	10.78	3.06

In order to investigate in depth frame performances, Incremental Dynamic Analyses (IDA) were carried out along the lines of Vamvatsikos and Cornell (2004) by means of the software IDARC-2D (Valles et al., 1996). The accelerograms used to perform these simulations were artificially generated with the SIMQKE software (Vanmarcke et al. 1990) in order to match Type 1 spectrum of Eurocode 8 (UNI EN 1998-1, 2005) for three different soil types as illustrated in Figure 9a and 9b, respectively.



a)



b)

Figure 9 a) Response spectra of accelerograms matching Type 1 Eurocode 8 spectra; b) Time history of an artificial accelerogram matching Type 1 Eurocode 8 spectrum B.

Several data were obtained from simulations as interstorey drifts, capacity curves, see for instance Figure 9, and forces in members, among others. For brevity, only behaviour factors are reported and listed in Table 5.2.

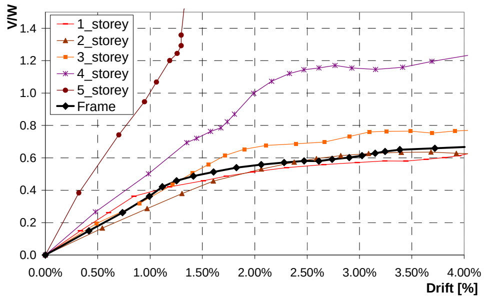


Figure 9: IDA capacity curves with soil Type D for a frame endowed with steel sheeting slabs.

Table 5.2. Behaviour factors from IDA analyses.

	Without connectors			With connectors		
	IDA A	IDA B	IDA D	IDA A	IDA B	IDA D
Steel sheeting slab	3.82	3.57	3.58	4.24	4.10	4.00
Prefabricated slab	3.17	3.42	3.48	3.75	3.94	5.03

From simulation results we can state that the analysed frames were capable of developing ductile mechanisms meeting the target displacement criteria set by Eurocode 8 (UNI EN 1998-1, 2005); a behaviour factor of around 4 was achieved; thus, Type 2 joint can be used in Ductility Class M structures (EN 1998-1, 2005); largest values of the damage index for the all studied frames were about 0.43 for exterior joints and 0.34 for interior joints:

thus the damage in joints was limited and can be considered repairable (Williams and Sexsmith, 1995).

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 joints with concrete filled tubes. In detail, the paper has focused on experimental results of beam-to-column joints both with steel sheeting and prefabricated slabs, together with numerical simulation results of moment resisting frames endowed with the proposed joints. In detail, results showed that the joints are rigid full-strength; consequently, energy dissipating mechanisms in moment resisting frames endowed with these joints will entirely rely on the formation of plastic hinges at adjacent beam ends. Moreover, a behaviour factor of around 4 was observed for the composite frames analysed. Because, relevant specimens subjected to cyclic loading exhibited plastic rotations greater than 25 mrad, the proposed joints can be deemed adequate for moment resisting frames of Medium ductility class (EN 1998-1, 2005).

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