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Refined numerical modelling for the structural assessment of steel-concrete composite beam-to-column joints under seismic loads

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ABSTRACT

This paper proposes a refined Finite-Element (FE) numerical approach to predict both global and local behaviour of steel-concrete composite welded joints subjected to seismic loads. The reference FE model is implemented in ABAQUS and first extensively validated to the full-scale experimental results of a welded steel-concrete composite specimen tested in a past research project, where the beam-to-column sub-assemblages were designed according to the prescriptions of Eurocode 4 and Eurocode 8. As shown, due to the FE modelling assumptions, a rather close agreement was generally found between the FE predictions and the corresponding test measurements, both in terms of global and local phenomena. Therefore, it is first expected that such numerical approach could be implemented as an alternative to costly and time consuming full-scale experimental tests, allowing an extensive parametric investigation of composite joints and possible design optimizations. An implicit advantage of the implemented FE model, in fact, is that according to a refined analysis of the experimental and numerical results for the welded joint object of investigation, the efficiency of the Eurocode 8 design prescriptions for steel-concrete composite joint details can be critically discussed and reviewed. In the specific case, a possible improvement of the design recommendations for the slab reinforcement around the column is proposed.

1. Introduction

The behaviour of composite joints is a major issue for steel-concrete composite moment resisting frames subjected to seismic loads; many studies [1–3] investigated their influence on the seismic response of frames considering different design strategies. The current design procedures are based on experimental results and numerical analyses carried out by various researchers mainly in the last 20 years.

The main results of tests on composite joints carried out until 1989 are summarized in [4], then further monotonic and cyclic tests have been carried out on various type of full-scale composite joint sub-assemblages representing exterior and interior beam to column connections [5–16].

The majority of tests past has been performed to study the stiffness, strength, ductility, and energy dissipation capacity of the sub-assemblages and emphasized the effects of deformation of the composite slab and the panel zone (e.g. [6]).

Simões et al. [8–9] focused on the identification of the concrete confinement contribution on composite columns, as well as on the assessment of strength and stiffness degradation. Experimental tests allowed then to study the effect of load path, identifying the corresponding failure modes and fitting the corresponding hysteretic curves by means of the Richard-Abbott and Mazzolani models [16–17].

Green et al. [10] showed that two failure modes must be considered in the design of a composite joint subjected to large cyclic bidirectional loads: a "punch through" of the column web due to force transfer from the "pinned" side of the connection and a crushing mechanism of the concrete slab in contact with the column. More generally, connections with dissipation occurring in the joints components have been tested to verify the influence of the concrete slab and its reinforcement around the column (e.g. [15]), the presence of a cantilever edge strip in exterior joints, the direction of the ribs of the steel sheeting, the presence of supplementary column web panel and stiffeners.

Finite-Element (FE) numerical models related to several experimental tests have been also proposed in the last years, in order to predict the global inelastic response (both monotonic and cyclic) of exterior and interior beam-to-column joints [18–22]. Anyway, the experimental tests on composite joints conducted so far and the

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corresponding FE numerical representations, although available in a large number in the literature, have been mainly focused on the prediction of the global behaviour only. Differing from existing research projects, this paper focuses on the FE numerical prediction of both global and local behaviour of steel-concrete composite joints.

To this aim, two full-scale exterior steel-concrete composite joints were designed according to the Eurocode 8 and 4 provisions [23,24] and characterized by the presence of two types of connections, i.e. a steel bolted extended endplate connection and a welded connection.

Cyclic experiments were then carried out on both specimens; see [25,26] for a description of the experimental program.

In this paper, as an intermediate stage of an extended research investigation, careful attention is paid to the FE numerical modelling of the welded connection only, which exhibited some relevant mechanisms during the past experiment. The specimen, tested under cyclic loading, was in fact extensively instrumented and a wide range of parameters was monitored during the experimental test. In this respect, the experimental setup and measurements are first recalled from [25].

Based on both global and local available test measurements, the consistency and accuracy of a refined Finite-Element numerical model is then proposed and critically discussed. The main advantage of the presented 3D FE numerical model, implemented in ABAQUS [27,28], is that the actual geometrical and mechanical properties of the joint components, as well as their reciprocal interaction, can be properly taken into account. Given the rather close agreement between experimental results and FE numerical predictions, in particular, it is expected that such numerical approach could be implemented as an alternative to costly and time consuming experimental tests, allowing an extensive parametric investigation of composite joints, including a wide set of geometrical and mechanical configurations for steel-concrete structural systems. An implicit advantage of the same FE modelling approach, as also discussed in this paper, is given in fact the critical discussion and possible review of the design recommendations actually provided by the Eurocode 8 for the steel-concrete composite joints.

2. Past experimental test

2.1. Specimen geometrical details and material properties

Through the FE exploratory investigation, the exterior full-scale composite welded joint (hereinafter called WJ) investigated in [25,26] was taken into account. As shown in Figs. 1–4, the WJ specimen was characterized by the presence of a 330 mm-long (from the external flange of the column) concrete cantilever edge strip and a 2000 mm-long (from the internal face of the column) IPE240 steel beam section directly welded to a 3400 mm-long HE280B partially encased composite column section. Table 1 collects the main features of the steel profiles that were taken into account for the design of the composite beam and column, at the time of the experimental research project.

The steel beam was connected with a 1000 mm-wide and 120 mm-thick concrete slab by means of two 100 mm × 19 mm diameter headed studs placed every 150 mm. The number of shear studs was calculated to ensure a full shear connection between the slab and the beam [24]. The arrangement of the shear studs is shown in Fig. 2(a) and (b). The slab was cast on a trapezoidal steel decking profile and reinforced with six U-shaped longitudinal rebars of 14 mm in diameter (Fig. 2(c)). Furthermore, three transversal rebars of 20 mm in diameter (Fig. 2(c)) were added in front of the column as per Eurocode 8 – Annex C [23].

The concrete region comprised between the column flanges was reinforced with four 12 mm-diameter longitudinal rebars, while 8 mm-diameter rebars welded to the column web were used as stirrups. The stirrups were 40 mm-spaced for a total length of 450 mm from the top and bottom flanges of the beam, and 150 mm-spaced elsewhere (Fig. 3(b)).

The partially encased composite column was pinned at both ends and 10 mm-thick transversal web stiffeners were added at the locations where the vertical load was applied (i.e. 1660 mm from the internal flange of the column) to prevent premature web buckling due to the concentrated load. 10 mm-thick full depth longitudinal stiffeners were also added at the beam-column intersection.

In terms of mechanical properties, the actual concrete compressive strength was characterized by means of standard tests carried out on two 300 mm-height and 150 mm-diameter cylinders. A mean cylindrical compressive strength f_c equal to 24.3 MPa was found (Table 2).

Tensile tests on three specimens were performed to find the actual resistance of the reinforcing bars and of the steel profiles. The average values of the yielding strength, ultimate strength and ultimate strain found for the steel components (rebars, steel flange and steel web) are shown in Table 2. Further details are available in [25].

2.2. Experimental setup, instrumentation and test procedure

As deeply discussed in [25], the cyclic loading history of the past experimental test was based on the ATC-24 protocol [29], see Fig. 5. In accordance with the test setup of Fig. 1, the normal force N was assumed null, while the cyclic vertical forces F (see Fig. 5) were considered as positive when upwards (i.e. sagging bending moment in the beam), and negative when downwards (i.e. hogging moment in the beam).

In order to obtain as much information as possible from each test, a combination of linear transducers and strain gauges was then used to record the displacements and strain in the main specimen components. The arrangement of the LVDTs is shown in Fig. 4 (a), while positioning of strain gauges ('sg', in the following) is shown in Fig. 4(b) and (c). A linear potentiometer was also used to measure the deflection at the end of the beam (Fig. 4a). Finally, to better explore the bending behaviour of the concrete slab, three transducers were placed on its top surface.

2.3. Experimental results

2.3.1. Global response

The collapse was reached due to the sudden local buckling of the flange in compression (Fig. 6) which exhibited a wavelength approximately equal to 240 mm, hence coinciding with the height of the steel section. The last cycle of the experiment was also characterized by lateral-torsional buckling due to extensive damage in the concrete slab.

The overall load-displacement cyclic response and the skeleton curves for the welded specimen are shown in Fig. 7. Table 3 summarizes the experimental results characterizing the global behaviour of the joint subjected to sagging and hogging moment, i.e. the displacement and load at yielding (d_y and F_y), the maximum load reached during the loading history and the corresponding displacement (F_{max} and d_{max}) and the ultimate displacement (d_u).

The yielding point was respectively identified with the yielding of the bottom flange of the steel beam under the sagging moment and the yielding of the rebars under the hogging moment [25]. Table 3 also reports the ductility of the connection, calculated as the ratio between the ultimate displacement (d_u) and the yielding displacement (d_y) of the joint. As shown, a ratio greater than 3 for