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Modeling of gravity-designed RC sub-assemblages subjected to lateral loads

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ABSTRACT

This paper aims to propose numerical models for seismic assessment of old existing RC beam-column sub-assemblages. In particular, two modeling approaches are described: for internal (usually named "C joints") and for external (named "T joints") beam-column sub-assemblages, both designed only for vertical loads without reinforcing details rules (such as inadequate bars lap splice, absence of hoops within the joint panel). They were developed starting from the failure mechanisms observed during an experimental campaign carried out on specimens reproducing old existing RC sub-assemblages cast in full and reduced scale, as well as reinforced with smooth longitudinal bars. The proposed models are not time-consuming with respect to both modeling and computational efforts, and do not require any calibration procedure from an experimental data set. They may be easily implemented in a general-purpose finite element program since merely based on geometrical and mechanical properties of RC elements. The numerical predictions show a good agreement with the experimental ones demonstrating that the proposed models are able of reproducing the lateral response of the two considered sub-assemblages typologies, in terms of both failure mechanism and hysteretic dissipative capacity.

1. Introduction

Starting from the two last decades, the engineering community is dedicating, a lot of effort in investigating the behavior of old existing reinforced concrete (RC) structures under seismic actions. The reason is owing to the fact that most of these structures result still in service nowadays in spite of they were designed only for vertical loads in seismic prone areas. As observed from the last earthquakes (Kocaeli, 1999; L'Aquila, 2009), their seismic response is very often inadequate since for moderate events, because characterized by local failure mechanisms with limited ductility (such as shear failure of joint panel and/or elements, strong beams-weak columns behavior) and also provoking a loss of vertical loads carrying capacity.

Many research groups have focused their work on the seismic assessment of old existing RC structures. Their attention has been primarily focused on laboratory simulations carried out on full-scale or reduced-scale sub-assemblages, replicas of parts of old moment resisting frames mainly designed for vertical loads and having a non-ductile behavior. Among others, for example, Hakuto et al. [15] and Liu and Park [17] conducted a series of laboratory tests, by imposing in the quasi-static way a lateral displacement history to poorly detailed RC internal and external beam-columns, typical of pre-1970s designed moment resisting frames built in New Zealand. Recently, Masi et al. [19] in his experimental program tested external RC beam-column joints reproducing connections designed in accordance with the 70s Italian design code [21], where also the influence of the acting axial load on the failure mode of the specimen was investigated. It is noteworthy that in this study seismically designed connections were tested, too. Experimental data on entire existing RC structures with substandard details may be found in Gilmartin et al. [14], where earthquake motion records were used to assess the structural and nonstructural response of the 7-stories Van Nuys Hotel, or in Calvi et al. [7], where on the contrary a prototype frame was tested in laboratory under lateral loads up to a significant damage level. These studies are also significant from an experimental standpoint because they allow us studying the interaction among the sub-assemblages local responses and the global response of an entire RC frame.

The occurred damages observation has revealed that the local failure mechanisms of the existing RC structures are strictly related to the absence of capacity design principles for structural ductility and/or to the lack of detailing rules, such as: scarcity of transverse reinforcement within the critical regions of beams and columns for

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restraining compressed longitudinal bars and confining section core; absence of hoops within the joint panel for shear; insufficient lap splice length of longitudinal bars. In these structures were usually used low-strength concrete and smooth bars too, provoking marked slippages, under seismic excitations, with a consequent drastic reduction of lateral stiffness and hysteretic energy dissipated. Confirmations of this may be found in Eligehausen et al. [11] and Braga et al. [4].

Many numerical models for simulating seismic collapse of old RC frame structures, using both detailed and/or reduced models, have been also proposed in literature by many researchers. These models may mainly be grouped into "*single*" or "*multi*" spring models, depending if the nonlinear response is completely lumped within rotational springs at element ends converging into the joint or not. For example, the models proposed by Otani [24], Alath and Kunnath [1], and Biddah and Ghobarah [2] belong to the *single-spring models* group. Central issue of these models, for a rational and accurate numerical simulation, is the definition of the nonlinear moment-rotation relationship to assign to springs at elements ends, also incorporating the longitudinal bars bond-slips effects. This requires very often a parameters calibration process established on a set of experimental results, making rather difficult the definition of a general nonlinear moment-rotation relationship accounting for all the various failure modes that may occur.

On the other hand, with the *multi-spring models* approach the joint panel is modeled as finite area connecting, with a set of elements, the converging beams and columns ends. This approach, more refined with respect to spring one, may be found for example into Lowes and Altoontash [18]and Mitra and Lowes [22], where a series of springs is implemented for the followings failure modes: the shear-panel springs to simulate the shear failure of the joint core; the bar-slip springs to simulate the anchorage failure of longitudinal reinforcements; and the interface-shear springs to simulate the shear-transfer failure at the joint-beam and joint-column interfaces. Again, also for these models the crucial issue for a realistic numerical failure behaviors prediction is the definition of the nonlinear constitutive law of each spring belonging to connecting joint panel area.

In according with these premises, numerical models for seismic assessment of old existing RC sub-assemblages are proposed in this paper. They refer to beam-column joints reinforced with smooth bars and belonging to typical RC buildings constructed in Italy and Mediterranean area during '50s–'70s. These models, falling into macro models group, do not require any calibration procedure depending on the experimental results and they may be directly implemented in a finite element software program merely starting from the geometrical and mechanical properties of the RC elements. The longitudinal bars bond-slips effects are incorporated at material level by using the modified steel model proposed and validated by the authors in previous works [5,10]. The proposed models herein presented are established and validated with an experimental campaign carried out on old existing sub-assemblage at the University of Basilicata [4]. Moreover, in this paper is also examined an internal joint that, after a first experimental test, has been repaired with FRP wraps applied to the column ends in the vicinity of joint panel and tested again. Finally, the numerical models are also validated with experimental results of other prototypes of existing RC sub-assemblages published in literature.

2. Test specimens and discussion of experimental results

In order to investigate the failure mechanisms under lateral loads of old existing RC concrete structures, an experimental campaign on beam-column sub-assemblages was performed. These specimens reproduced internal (named "C" joints) and external ("T" joints) beam-column connections of a non-ductile prototype of RC frame (Fig. 1) built in Italy during the '50s–'70s, and consisting of 3 floors and 3 spans. Specimens were cast in full scale and in scale 2:3, and designed only for gravity loads accordingly with the old design practice adopted in Italy [25] in the past.

All beams and columns were reinforced with smooth bars and poorly detailed within end regions converging into the joints, and no transverse reinforcement was provided within the joint cores.

As for sub-assemblages in scale 2:3, the columns and beams had cross-sectional dimensions of 200 mm × 200 mm and 200 × 333 mm, respectively, whereas the cross-sectional dimensions of columns and beams of the specimens in scale 1:1 were 300 mm × 300 mm and 300 mm × 500 mm. All geometrical and material details on the specimens may be found elsewhere [4].

In this study the experimental results of two different sub-assemblages are discussed in detail: the internal joint built in scale 1:1 and named "C11" joint, and the external joint built in scale 2:3 and named "T23" joint. The reinforcements and section details of both specimens are reported in Fig. 2 and listed in Tables 1 and 2. It should be noted that the longitudinal reinforcements of the specimens are continuous throughout the panel joint.

In Tables 3 and 4 the specimens material strengths are summarized. In particular, uniaxial tensile tests were conducted for each bar diameter in order to determine the yielding and the ultimate strength, and the fracture strain referred to 10 diameters length (Table 3). The steel corresponded to mild steel in according to R. D. '39, also classified as Aq. 42 steel. As regards the compressive strength mean value of concrete, it was obtained by testing cubic concrete specimens of 15 cm side after 30 days curing (Table 4).

The tests were carried out in vertical position by imposing an increasing lateral displacement at top of upper column in the quasi-static way reproducing, for each sub-assemblage investigated, the same deformed shape that it would have into the entire frame if subjected to lateral actions. The displacements time-history was controlled by an actuator of +490 kN/−290 kN connected by hinges to a RC contrast wall and to the upper column top, and it consisted of groups of three cycles with an increasing magnitude and constant rate equal to 1 mm/s and 1.5 mm/s for the scaled 2:3 joints and for full-scale specimens, respectively. At the top of the upper column was applied a vertical load kept

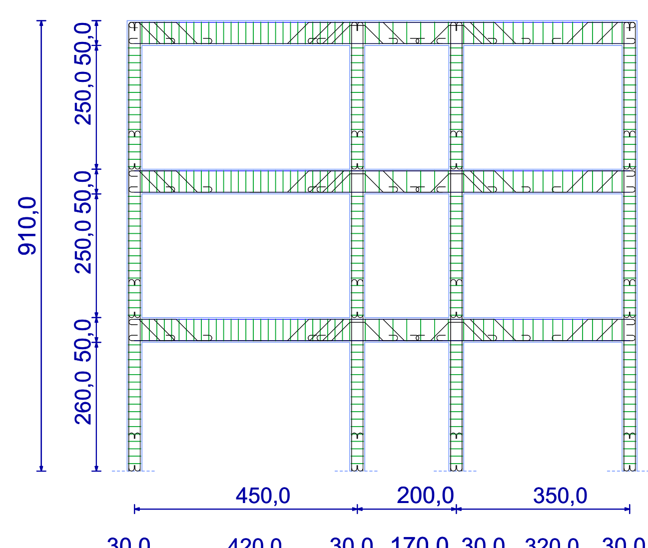

Fig. 1. Prototype RC frame, designed accordingly with the old design practice R.D. 1939. Dimensions are in millimetres.