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Experimental assessment of unreinforced exterior beam–column joints with deformed bars

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ABSTRACT

In the performance assessment of typical existing buildings, seismic collapse safety might be significantly affected by non-linear behavior of joints that are involved in the failure mechanism, especially if they are characterized by poor structural detailing such as the lack of an adequate transverse reinforcement in the joint panel.

Unfortunately, commonly accepted tools to assess existing joints capacity are not available. Few reliable approaches for modeling all sources of nonlinearity are proposed in literature for poorly designed beam–column joints because of relatively poor information from experimental tests.

The present study aims at improving the understanding of seismic performance of exterior joints without transverse reinforcement in existing RC buildings through experimental tests.

Two full-scale exterior unreinforced beam–column joint sub-assemblages are tested under cyclic loading. The specimens are reinforced with deformed bars but they are different for beam longitudinal reinforcement ratio. Two different kinds of joint failure are expected, with or without the yielding of the adjacent beam. Strain gauges located on beam bars and displacement transducers on the joint panel allow the complete definition of both the main deformability contributions, namely fixed-end-rotation and shear strain of joint panel, highlighting the differences between failure modes.

Design criteria, adopted setup and experimental results are described and discussed. Finally, experimental results are compared with proposals from literature in terms of shear strength.

1. Introduction

Reinforced Concrete (RC) buildings designed for gravity loads only or according to obsolete seismic codes are widespread in Italian and Mediterranean building stock. For these buildings, beam–column joints represent a critical issue; the lack of capacity design principles leads to a low shear strength of the joint, potentially leading to a shear failure that limits the deformation capacity of adjoining beams and/or columns [1,2].

Past earthquakes showed that shear failure in beam–column joints can lead to building collapse [3], which often can be attributed to inadequate joint confinement. In recent earthquakes (e.g., Izmit 1999 [4], Tehuacan 1999 [5], Chi-Chi 1999 [6]), the inadequacy of building joints designed according to older standards was one of the main causes of severe damage or collapses. In particular, the observation of damage after L'Aquila earthquake (2009) indicated that RC buildings designed in Italy before the mid-1990s may have serious structural deficiencies especially in joint regions, mainly due to a lack of capacity design approach and/or poor detailing of reinforcement [7].

A significant amount of experimental research about seismic performance of RC beam–column joints has been carried out in the last forty years. The majority of the research literature has emphasized the improvement of the performance of RC beam–column joints through new design concepts and improved details, such as joint hoops or improved anchorage. Only in last years, an increasing interest about the analysis of unreinforced beam–column joints developed. In particular, experimental research is focused above all on exterior unreinforced joints [8], mainly due to the higher seismic vulnerability of this joint typology with respect to interior joints [9]. Two main goals are pursued in these experimental studies: (i) to assess seismic performance of unreinforced beam–column joints in "as-built" condition (for instance [10–13]); (ii) to evaluate the effectiveness of possible retrofitting strategies, such as the adoption of fiber-reinforced polymer materials [14–16], RC or steel jacketing [17–19], or post-installed anchors [20].

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In literature [21–24], comprehensive experimental databases of unreinforced exterior RC joints have been presented and discussed. In most of these tests, researchers focused their attention on joint shear strength and on the influence of several key parameters on this strength. In particular, generally investigated parameters are geometric parameters (for instance, the joint aspect ratio) or mechanical parameters (such as axial load ratio, concrete compressive strength, and longitudinal reinforcement ratio). Vice-versa, the number of experimental studies focused on (i) the main deformability sources due to joint, (ii) their contribution to the overall deformability, or (iii) local shear response of the joint panel is quite smaller. Nevertheless, a complete characterization of all the deformability contributions due to joints is paramount to understand the role of beam–column joints, not only in terms of strength, on nonlinear response of RC frames. Furthermore, a complete evaluation of joint local response allows properly modeling these elements into structural models for analysis of non-conforming RC frames.

This study aims at improving the understanding of seismic performance of unreinforced exterior joints in existing RC buildings, investigating on the influence of longitudinal reinforcement ratio on the joint shear strength, on one hand, and on the deformability contributions ascribable to the joint region on the response of the sub-assemblages, on the other hand. Two experimental tests on unreinforced exterior joints, without transverse beam, are tested under cyclic loading. The joint specimens are designed according to code prescriptions and design practices in force in Italy between 1970s and 1990s. Two distinct failure modes are expected: a joint shear failure prior to beam yielding (hereinafter referred to as J failure mode) and a joint shear failure following beam yielding (hereinafter referred to as BJ failure mode). The global experimental response and the evolution of observed damage are presented. The main deformation mechanisms of the RC joint region are discussed, and their contribution to the overall deformability is investigated. Moreover, local shear stress–strain response of joint panel is evaluated. Finally, main joint shear strength models existing in literature or codes are compared with experimental results.

The tests presented herein can provide a useful contribution to enhance the quite poor experimental database of unreinforced exterior joints, with respect to the corresponding database of reinforced exterior joints [8], and to better investigate about local response, not always analyzed in tests from literature.

2. Experimental program and setup

2.1. Specimens description

Two full-scale exterior unreinforced beam–column joint sub-assemblages (Fig. 1) have been tested under cyclic loading. The two tests are identical for geometry. Beam rectangular sectional area is 50 cm high (h_b = 50 cm) and 30 cm depth (b_b = 30 cm). Columns (top and bottom) have a square sectional area with height (h_c) equal to 30 cm. Column length was designed to be representative of typical interstorey height (3.40 m), and column shear length (L_c) is equal to 1.45 m. The beam length (up to the centerline of the column) is equal to 1.80 m, and its shear length is L_b = 1.65 m.

As shown in Fig. 1, in Test #1 the beam is symmetrically reinforced with 4 ϕ 20 bars for both reinforcement layers (corresponding to compression and tension reinforcement ratios equal to $\rho' = \rho = 0.84\%$); also the column is symmetrically reinforced with 4 ϕ 20 bars in top and bottom layers, corresponding to a total reinforcement ratio ($\rho' + \rho$) equal to 2.79%.

In Test #2, the beam is symmetrically reinforced with 4 ϕ 12 bars (corresponding to compression and tension reinforcement ratios equal to $\rho' = \rho = 0.30\%$); also the column is symmetrically reinforced with 4 ϕ 12 bars, corresponding to a total reinforcement ratio ($\rho' + \rho$) equal to 1.01%.

In both cases, ends of top and bottom beam longitudinal bars are bent at 90° into the joint core for a length of 20 cm. The transverse reinforcement consists of 8 mm diameter closed stirrups with both ends bent at 90° and 10 cm long. The stirrups are spaced at 10 cm along the beam and the column except within 62 cm of beam and column end, where the spacing is reduced to 5 cm to give adequate strength at the location where forces are applied during the test. The longitudinal reinforcement in the column extends continuously up through the joint from the bottom to the top of the column. The test unit reinforcement cages were constructed as shown in Fig. 1 and cast in place horizontally (see Fig. 2). A high-frequency vibrator was used to consolidate the concrete. Each test unit was allowed to cure for at least 72 h before they were removed from the forms.

2.2. Materials

Concrete compressive strength for all specimens was evaluated on four $15 \times 15 \times 15$ cm^3 cubic samples (CSs) of the casted concrete. Values of 28-day cylindrical strength for each CS and their mean value are reported in Table 1. Commercial typology of the adopted reinforcing steel is B450C, i.e., class C reinforcement with f_{yk} = 450 MPa according to Annex C provisions of Eurocode 2 (EN 1992-1-1:2004 – Annex C) [25]. Steel typology B450C shows mechanical properties that can be assimilated to FeB44k typology, widespread in Italy between 1970s and 1990s. Tensile tests were carried out on three samples for each bar diameter. Table 2 reports mean values of their mechanical properties, namely yield strength (f_y), ultimate strength (f_t) and hardening ratio (f_t/f_y).

2.3. Design of specimens

Geometry and longitudinal reinforcement in beam and columns are defined by means of a simulated design procedure [26] of a perimeter 4-storey 5-bay frame according to code prescriptions and design practices in force in Italy between 1970s and 1990s. In particular, the analyzed specimens are intended to be representative of exterior joints of the first floor of such a frame. The specimen named Test #1 is related to a frame designed according to seismic prescriptions (for high seismicity level), in compliance with the Italian codes [27–29]. The specimen named Test #2 is related to a frame designed for gravity loads only.

Beam longitudinal reinforcement ($A_{s,b}$) is defined on the basis of flexural demand obtained from the simulated design, assuming an allowable stress for steel equal to 220 MPa (corresponding to a steel typology named FeB38k or FeB44k, commonly adopted in Italy in 70s–90s). Minimum amount of longitudinal reinforcement required in columns is slightly modified with respect to the simulated design and a weak beam–strong column hierarchy is obtained. Column axial load corresponding to gravity loads only is equal to 260 kN.

Transverse reinforcement in beam and columns was designed to avoid shear failure, in order to not preclude joint shear failure; whereas no transverse reinforcement was located in the joint panel zone, in compliance with code prescriptions in force in the reference time assumed for design.

Based on the adopted design practice and material mechanical properties, two failure modes are expected: a joint shear failure prior to beam yielding, for Test #1, and a joint shear failure following beam yielding, for Test #2.

According to ASCE/SEI 41 [9] a joint shear strength ($V_{jh,max}$) equal to 241.5 kN has been calculated, corresponding to a joint shear stress (τ_j) equal to 2.68 MPa. On the other hand, joint shear