CAPACITY MODELS OF BEAM-COLUMN JOINTS: PROVISIONS OF EUROPEAN AND ITALIAN SEISMIC CODES AND POSSIBLE IMPROVEMENTS

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ABSTRACT
More reliable assessment procedures of existing RC buildings are currently available, and have been introduced in the Italian and European codes reporting new rules for seismic design and analysis. However, further studies are necessary in order to upgrade such procedures and, specifically, to test the effectiveness of the capacity evaluation methods relevant to beam-column joints. To this purpose, a literature review on the subject and a wide experimental program on exterior beam-column joint specimens were carried out in the framework of the DPC-ReLUIS Project (Research Line 2, Task NODI). Some results are reported in the present paper to highlighting the role of the key behavioural parameters of RC beam-column joints, thus giving useful suggestions on the reliability of current code expressions and on possible improvements.

KEYWORDS: existing buildings, reinforced concrete, beam-column joint, capacity model.

1 INTRODUCTION

In the framework of DPC-ReLUIS 2005-2008 Research Project, a Research Line (RL 2) was devoted to the “Assessment and Reduction of Seismic Vulnerability of RC Existing Buildings”. In this RL, a task was specifically devoted to the “Behaviour and Strengthening of Beam-Column Joints”. In fact, in spite of a more reliable assessment of this type of structures, and simultaneously with the promulgation of new rules for seismic design and analysis in Italy and Europe, further work is necessary to evaluate the effectiveness of the capacity evaluation methods relevant to beam-column joints contained in the codes. To this purpose, the work was firstly devoted to an accurate literature review on the available capacity evaluation methods, pointing out the most appropriate ones to the Italian and European building stock. Then, such capacity models have been applied to test results deriving from experimental programs on beam-column joints reported in the technical literature, highlighting the models able to better predict the experimental results.
As for the ReLUIS experimental program, although the total number of tests performed is not so high and relevant only to exterior joints, useful indications have been derived on the prediction capability of code expressions and in order to carefully select the design and verification parameters. This can be made comparing the real behaviour in beam-column joints detected during experimental tests, with respect to the conventional performances determined strictly applying the codes.

2 OVERVIEW OF CURRENT RESEARCH STATE

2.1 Capacity models

A reliable evaluation of strength and deformability of beam-column joints is a crucial aspect in the framework of performance-based design or evaluation of Reinforced Concrete (RC) buildings, as confirmed by recent experimental activities and damage observations from last earthquakes. Nowadays, a large consensus has not been found on a single joint modeling technique neither in the scientific literature nor in the Codes, in spite of the fact that many research groups worldwide, during the last three decades, performed wide experimental and theoretical studies on this topic to evaluate the cyclic behavior of beam-column joints. As shown by many experimental programs, the failure of joint panels is induced usually by shear or bond flaws. The stress distribution due to flexural and shear forces transferred through the joint produces a wide diagonal crack pattern in the panel leading to a crushing failure of the compressed strut and consequently to strength and stiffness deterioration. The cyclic deterioration of bond performance, on one side, yields to reduced flexural strength and ductility of framing elements (Hakuto et al., 1999; Manfredi et al., 2008), while it yields, on the other, to a noticeable increase in the story drift (Soleimani et al., 1979).

\[ V_{jh} = C_{s2} + C_{c2} + T_i - V' = T_i + T_2 - V' \]  \hspace{1cm} (2.1.1)

\[ C_{s2} + C_{c2} = T_2 \]  \hspace{1cm} (2.1.2)

In Figure 2.1.1(a) the forces acting on an interior beam-column joint panel are reported. The horizontal shear force is equal to:

\[ V_{jh} = C_{s2} + C_{c2} + T_i - V' = T_i + T_2 - V' \]  \hspace{1cm} (2.1.1)

because the horizontal equilibrium equation referred to the end section of the beam yields to:

\[ C_{s2} + C_{c2} = T_2 \]  \hspace{1cm} (2.1.2)
The vertical shear force can be similarly given by an equilibrium equation, but an accurate evaluation can be given by:

\[ V_{jh} = \frac{h_b}{h_c} V_{jh} \quad (2.1.3) \]

where \( h_b \) is the beam depth and \( h_c \) is the column depth.

The shear transfer mechanisms allowing for the joint force transfer, after the diagonal cracking of the joint panel, are shown in figure 2.1.1(b-c) (Paulay et al., 1992). In the first mechanism, named strut-mechanism, the joint shear is concentrated in a single compressed concrete strut. In this case the transverse steel reinforcement provides confinement to the concrete allowing for higher deformability of the strut, but only before steel yielding. In the second mechanism, named truss-mechanism, the portion of the shear force due to the bond stress along the longitudinal steel reinforcement inside the joint is in equilibrium with a truss mechanism given by concrete struts and vertical and horizontal ties corresponding to joint panel reinforcement. The shear capacity is given by the sum of the shear contributions according to these two mechanisms. It is worth noting that if bond deterioration occurs, the shear contribution due to the strut-mechanism increases, while the total shear force acting on the joint panel remains constant. Assuming a full deterioration of the bond capacity between steel bars and concrete, it is:

\[ C_{s2} = -T_1 \quad (2.1.4) \]

so that the following equation is given, that is equal to (2.1.1):

\[ V_{jh} = C_{s2} + C_{c2} + T_1 - V_c = -T_1 + T_1 + T_1 - V_c = T_1 + T_2 - V_c \quad (2.1.5) \]

A first approach to evaluate the shear capacity of a beam-column joint without transverse reinforcement consists in principal stress limits according to concrete strength. Direct limits on the shear stress, according to (Hakuto et al., 2000) and reported in many International Codes (ACI 352, 2002; AIJ, 1999) are not accurate because they do not account for the vertical axial force in the column. The principal stresses in the joint panel can be given by Mohr’s Circle assuming uniform normal and transverse stresses, \( f_a \) e \( v_{jh} \) respectively, according to the following equation:

\[ p = \frac{-f_a}{2} \pm \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} \quad (2.1.6) \]

Equation (2.1.6) allows the horizontal joint shear to be evaluated at the first development of diagonal cracking:

\[ v_{jh} = p_1 \sqrt{1 + \frac{f_a}{p_1}} \Rightarrow V_{jh} = k_1 f_c \sqrt{1 + \frac{f_a}{k_1 f_c}} b_j h_j \quad (2.1.7) \]

where the tensile limit stress is assumed to be proportional to \( k_1 \) times the square root of concrete compressive strength, where \( k_1 \) is empirically evaluated. It is clearly shown in (2.1.7) that axial load delays the diagonal cracking in the joint panel.

The joint shear causing compressed concrete strut crushing, assuming the compressive limit stress to be proportional to \( k_2 \) times the concrete compressive strength, is equal to:

\[ v_{jh} = p_2 \sqrt{1 - \frac{f_a}{p_2}} \Rightarrow V_{jh} = k_2 f_c \sqrt{1 - \frac{f_a}{k_2 f_c}} b_j h_j \quad (2.1.8) \]

It is worth noting that a joint failure criterion based on tensile principal stress limit results to be over conservative. In fact the joint panel is able to transfer noticeable shear forces also in a cracked phase due to the diagonal strut mechanism. The joint failure should be in fact always
related to the compressed strut crushing. In the case of high axial loads, the compressed strut crushing can be attained before the joint panel cracking (Paulay et al., 1992).

The evaluation of the horizontal joint shear determining the compressed strut crushing, according to equation (2.1.8), needs the experimental evaluation of \( k_2 \) coefficient, accounting for the real stress field in the joint, that is complex to be evaluated in cracked phase, for the compressive strength deterioration due to diagonal tensile strains (Collins et al., 1980), and for the detailing of steel bars anchorage, that is a crucial aspect in the case of exterior joints, to guarantee the development of the compressed strut (Priestley, 1996).

In the case of interior joints without transverse reinforcement, the values for \( k_1 \) and \( k_2 \) (Priestley, 1996) are 0.29 and 0.50, respectively. In the case of exterior joints, the proposed value for \( k_1 \) according to Priestley depends on the longitudinal bars anchoring details and it is 0.29 if the longitudinal bars are bent at 90° outside the joint or 0.42 if they are anchored inside the joint. In the case of exterior joints with smooth bars the value for \( k_1 \) equal to 0.20 is suggested (Calvi et al., 2002). In the cited works, deterioration models are provided to account for \( k_1 \) and \( k_2 \) variability based on ductility demand.

The strength capacity of joints with transverse reinforcement can be evaluated according to cited model (Paulay et al., 1992). The minimum horizontal and vertical reinforcement requirements are reported here, details on the complete model and on the simplified assumption on shear partition in the two mechanisms are given in the original paper. In the case of interior joints, it is:

\[
A_{j_h} = \left(1.15 - 1.3 \frac{N_{col}}{f_c A_g}\right) \frac{\lambda_0 f_y}{f_{yh}} A_s; \quad A_{j_v} \geq \frac{1}{f_{jy}} \left(0.5V_{jv} - N_{col}\right) \quad (2.1.9)
\]

while in the case of exterior joints, it is:

\[
A_{j_h} = \beta \left(0.7 - \frac{N_{col}}{f_c A_g}\right) \frac{f_y}{f_{jh}} A_s; \quad A_{j_v} \geq \frac{1}{f_{jy}} \left(0.5V_{jv} - N_{col}\right) \quad (2.1.10)
\]

where \( \beta = A'_s/A_s \) is the tensile over compressed beam reinforcement ratio, \( f_c \) is the concrete compressive strength, \( N_{col} \) is the minimum axial load in the column and \( A_g \) is the column cross sectional area.

Many other models are available in literature (Sarsam et al., 1985; Vollum et al. 1999; Bakir et al., 2002) to evaluate the shear strength as the sum of concrete and steel reinforcement contributions. However such models are based on experimental calibration, while the model according to Paulay is based on force equilibrium. In this respect, even though sometimes these models seem to be more accurate, it is obvious that their reliability depends on the extent and completeness of the experimental database of tests used for their calibration.

During last years, many other works proposed different models to evaluate the strength and deformability of beam-column joints. Amongst the well known models, there is the model (Pantazopolou et al., 1992) based on simple equilibrium equations and extended by (Antopoulos et al., 2002) to FRP strengthened joints. Strut-and-tie models (Hwang et al., 1999; Mitra, 2007) are well known also to analyze the joint panels with or without transverse reinforcement. The “Quadruple flexural resistance in reinforced concrete beam-column joints” (Shiohara, 2001) model accounts for the equilibrium of four rigid bodies forming the joint panel. It allows the different failure modes to be determined accounting for bond behavior of reinforcement in exterior or interior joints. Amongst the models accounting for the inelastic cyclic behavior of beam-column joints and those based on experimental calibration of plastic hinges and/or rotational springs, it can be cited the macro-model proposed in (Lowes et al., 2004) to account for the beam-column joint behavior and implemented in the structural code OpenSees. This algorithm is not time-consuming, but it allows to reliably and objectively account for the main mechanisms determining the inelastic
cyclic behavior of the joints: namely, anchorage failure of the longitudinal reinforcement both in the columns and beams, shear failure of the joint panel and failure of the shear transfer mechanism at the joint interfaces.

2.2 Experimental results from the scientific literature

The mentioned models for the shear capacity of RC joints can be assessed by comparing the theoretical results with the experimental data derived from the scientific literature purposely collected in a wide database. In particular, 87 results of experimental tests have been considered in this database (Table 2.2.1): 66 tests were carried out on specimens with stirrups within the joint panel, while the remaining 21 tests deal with unreinforced RC joints. The former ones basically reproduce the behaviour of joints in new seismically designed structures while the latter ones aim at reproducing the response of existing RC members. As far as the loading modality, a large part of the tests were carried out under cyclic conditions.

<table>
<thead>
<tr>
<th>Authors</th>
<th>No. of tests</th>
<th>Authors</th>
<th>No. of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hwang &amp; Al. (2005)</td>
<td>5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The four diagrams in Fig. 2.2. and 2.2.2 point out the comparison between the experimental results in terms of shear strength with respect to the corresponding results derived by applying four of the above mentioned theoretical models. The experimental values of shear strength of the joint panel ($V_{Rj}^{exp}$) can be directly derived by the ultimate force applied on the beam through simple equilibrium equations.

![Graph a) Paulay & Priestly (1992) model](image1)

![Graph b) Sarsam & Phillips (1985) model](image2)

**Fig. 2.2.1. Shear capacity of RC joints: comparison of results from models and experimental tests.**
Fig. 2.2.2. Shear capacity of RC joints: comparison of results from models and experimental tests.

Fig. 2.2.a is devoted to the model by Paulay & Priestley (1992) resulting in generally scattered and often unconservative prediction of the experimental values. On the contrary, the model by Sarsam & Phillips (1986) is generally more conservative, albeit resulting in significant dispersion with respect to the experimental values (Fig. 2.2.b). The model by Vollum & Newmann (1999) results in a more precise prevision of the experimental results (Fig. 2.2.a), while the model by Bakir & Boduroglu (2002) is generally unconservative, yet highly correlated to the experimental results (Fig. 2.2.b).

3 CAPACITY EVALUATION IN SEISMIC CODES

As for the European Code EC8, both for new (CEN, 2004) and existing (CEN, 2005) buildings, the evaluation of the horizontal maximum shear acting in the joint panel (shear demand) can be performed through the following two expressions, respectively for exterior and interior joints:

\[
V_{jhd} = \gamma_{Rd} \cdot A_{s1} \cdot f_{yd} \cdot V_c - V_{c} 
\]

(3.1)

\[
V_{jhd} = \gamma_{Rd} \cdot (A_{s1} + A_{s2}) \cdot f_{yd} - V_{c} 
\]

(3.2)

where \(A_{s1}\) is the area of the beam top reinforcement, \(A_{s2}\) is the area of the beam bottom reinforcement, \(V_c\) is the column shear force, obtained from the analysis in the seismic design situation, \(\gamma_{Rd}\) is a factor to account for overstrength due to steel strain-hardening and should be not less than 1.2.

The EC8 formulation for predicting the joint shear capacity is made up of two separated steps. Firstly, there is an expression to evaluate the compression capacity of the strut that can be recognized in the joint panel under seismic actions and, then, an expression devoted to verify the tensile strength of the joint in order to avoid diagonal cracking.

The horizontal shear demand should not exceed a value that could cause the compression failure of the joint:

\[
V_{jhd} \leq \eta f_{cd} \sqrt{1 - \frac{V_d}{\eta b_j h_{jc}}} 
\]

(3.3)

where \(\eta = 0.60 (1-f_{ck}/250)\) for interior joints and \(\eta = 0.48 (1-f_{ck}/250)\) for exterior joints, practically meaning that the strength of exterior joints is 0.8 (0.48/0.60) times that one of
interior joints (assuming the same joint materials and detailing); \(v_d\) is the normalised axial force in the column above the joint, \(f_{ck}\) is given in MPa, \(h_{jc}\) is the distance between the extreme layers of column reinforcement, \(b_j\) is the effective width of the joint. Further, EC8 provides an expression to evaluate the joint transverse reinforcement (left hand term in (3.4)) needed to avoid the diagonal cracking caused by the achievement of the concrete tensile strength \(f_{ctd}\), as follows:

\[
\frac{A_{sh} \cdot f_{yd}}{b_j \cdot h_{jw}} \geq \left( \frac{V_{jhd}}{f_{ctd} + v_d f_{ctd}} \right) - f_{ctd}
\]

where, \(A_{sh}\) is the total area of the horizontal hoops in the joint, \(V_{jhd}\) is the horizontal joint shear demand, \(h_{jw}\) is the distance between top and bottom reinforcement of the beam. The Italian Code IC (Ministry of Infrastructures, 2008) deals separately with joints belonging to new and existing buildings, the former ones being evaluated as in EC8. As for existing buildings, IC contains two expressions devoted to verify beam-column joint without seismic provisions, that is without hoops in the panel (paragraph C8.7.2.5). These expressions allow to calculate the maximum diagonal compression (3.5) and tensile (3.6) stresses in the concrete joint core that must be below given values related to the concrete strength \(f_c\):

\[
\sigma_w = \frac{N}{2A_g} + \sqrt{\left( \frac{N}{2A_g} \right)^2 + \left( \frac{V_n}{A_g} \right)^2} \leq 0.5f_c \quad (3.5)
\]

\[
\sigma_n = \frac{N}{2A_g} - \sqrt{\left( \frac{N}{2A_g} \right)^2 + \left( \frac{V_n}{A_g} \right)^2} \leq 0.3\sqrt{f_c} \quad (f_c \text{ in MPa}) \quad (3.6)
\]

where \(N\) is the axial force acting on the upper column, \(A_g\) is the gross area of the joint panel horizontal section and \(V_n\) the horizontal shear acting in the joint panel evaluated accounting both the column shear and the shear transmitted by the beam reinforcing bars.

4 CRITICAL REVIEW OF CODE PROVISIONS

Results about joint behaviour obtained from the experimental tests carried out during ReLUIS project and further researches provided some useful information to critically review and analyse some expressions reported in the European Code EC8 and in the Italian Code IC.

4.1 Evaluation of the shear demand \(V_{jhd}\)

Expressions (3.1) and (3.2), regarding exterior and interior joints, respectively, may underestimate the shear demand that the beam can really transmit to the joint. Indeed, the amplified tension that appears in the expressions, \(\gamma_{fd}f_{yd}\), is a design value that the steel certainly exceeds when the plastic hinge develops in the beam.

By considering that:

(i) the real yielding strength \(f_y\) usually exceeds the nominal yielding strength \(f_{yk}\), even though it should not exceed over the 25% (EC8 par. 5.5.1.1(3)P and IC Tab. 11.3.1b), and

(ii) the characteristic value of the ratio between the ultimate strength \(f_u\) and the yielding strength \(f_y\) must be in the range 1.15-1.35,

it can be considered that steel bars, in case of large strains, can reach, on the average, a tension of the order of \(f_{ym} = 1.45 f_{yk}\) instead of the value suggested by EC8 and IC that is \(f_{yk}\) multiplied by \(\gamma_{yu} = 1.20\) and divided by \(\gamma_s = 1.15\) equal to 1.04 \(f_{yk}\).

An experimental proof of this fact has been obtained for the exterior joints tested by the Research Unit (RU) of University of Udine (Russo et al., 2007 and 2008) during the ReLUIS project, where the acting shear was calculated through the expression (3.1) using:
since only the mean value $f_{y,mean}$ of reinforcing steel was available and the characteristic value $f_{yk} = f_{ynom}$ was unknown. The values of $V_{jhd}$ calculated as described above and the maximum ones obtained by experimental results, $V_{exp}$, are significantly different, as shown in table 4.4.1.

Table 4.1.1. Calculated ($V_{jhd}$) and measured ($V_{exp}$) shear values.

<table>
<thead>
<tr>
<th>Joint</th>
<th>$V_{jhd}$ [kN]</th>
<th>$V_{exp}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>12-6 (+)</td>
<td>50.4</td>
<td>99.3</td>
</tr>
<tr>
<td>12-6 (-)</td>
<td>49.6</td>
<td>103.2</td>
</tr>
<tr>
<td>12-8 (+)</td>
<td>50.7</td>
<td>97.6</td>
</tr>
<tr>
<td>12-8 (-)</td>
<td>48.7</td>
<td>110.0</td>
</tr>
<tr>
<td>16-6 (+)</td>
<td>49.0</td>
<td>108.4</td>
</tr>
<tr>
<td>16-6 (-)</td>
<td>90.6</td>
<td>147.2</td>
</tr>
<tr>
<td>16-8 (+)</td>
<td>52.1</td>
<td>108.9</td>
</tr>
<tr>
<td>16-8 (-)</td>
<td>88.5</td>
<td>162.2</td>
</tr>
</tbody>
</table>

(+): positive acting moment, (-): negative acting moment

The $V_{exp}$ values shown in table 4.4.1 have been determined using the expression (3.1) where $\gamma_{Rd}A_{s}f_{yd}$ has been substituted by the actual force value provided by the reinforcing bars. This value is obtained by dividing the maximum experimental moment at the beam-column interface for $0.9d_b$, with $d_b$ the effective depth of the beam. Finally, $V_c$ is the maximum shear in the column corresponding to the maximum experimental beam moment.

From the above considerations and on the basis of the results shown in table 4.4.1, it can be deduced that the expressions (3.3) and (3.4) (either 5.33 and 5.35 of EC8 or 7.4.8 and 7.4.10 of IC) for resistance verification (maximum diagonal compression and tension in concrete core) of joints with or without hoops can be not conservative, being the acting shear $V_{jhd}$ (Eq.(3.1)) underestimated.

In addition, the calculation of demand in EC8 for joints of existing buildings is the same as for new buildings applying the equations (3.1) and (3.2), with the difference that, as reported in (CEN, 2005), the mean values of material resistance must be divided by the Confidence Factor (CF) and, in case of brittle elements as the beam-column joints are, by the partial factor $\gamma_s$. This can lead to a greater underestimation (with respect to the joints of new buildings) of the shear demand, using a very low tension in beam reinforcing bars equal to:

$$(4.1.2) f_{yd} = \gamma_{Rd} \cdot \frac{f_{ym}}{CF \cdot \gamma_s}$$

where $f_{ym}$ is the mean value of the yielding strength of steel.

Use of CF in expression (4.1.2) does not appear correct, as it should provide lower values of $f_{yd}$ to be used in equations (3.1) and (3.2), thus lower demand values on joints, at decreasing knowledge levels. It is, then, advisable, as typically suggested when strength values need to be used in calculating action effects delivered to brittle component/mechanism by ductile components, mean values of material properties are multiplied by the confidence factor in order to appropriately account for the attained knowledge level. Moreover, the safety factor $\gamma_s$ should be assumed equal to 1.0, thus expression (4.1.2) can be modified as follows:

$$(4.1.3) f_{yd} = \gamma_{Rd} \cdot CF \cdot f_{ym}$$

where the term $\gamma_{Rd} \cdot CF$ could be limited, for example, to the value of 1.45 taking into account what above reported.
5 ANALYSIS OF PREDICTION ABILITY OF CODE FORMULATIONS THROUGH ReLUISEXPERIMENTAL TESTS

In order to verify the estimation capability of the joint shear strength provided by the expressions contained in IC and EC8, they have been applied to the specimens tested at Laboratory of Structures of the University of Basilicata (UniBas RU) in the framework of the DPC-Reluis Project.

During the experimental program, 10 quasi-static cyclic tests on exterior full scale beam-column joints, provided with different Earthquake Resistant Design (ERD) Level, axial force and type of steel reinforcement, have been carried out. The main characteristics of the specimens under test and of the obtained results are summarized in table 5.1. The specimens could have three different Earthquake Resistant Design levels: design for seismic zone 2 (Z2), for seismic zone 4 (Z4) and with respect to gravity loads only (NE). The normalized axial load applied during test was equal either to 0.15 (NL) or 0.30 (NH). More details on the experimental program are reported in (Masi et al., 2008), (Masi et al., 2008b) and in (Masi & Santarsiero, 2008).

As it can be seen in table 5.1, 7 out of 10 specimens showed a failure mechanism that involved only the beams because of their small amount of longitudinal reinforcement, particularly the bottom one. Adopting the expressions (3.3)-(3.6), joint strength values (compression and tension capacities) have been determined for each of the 10 tested joints accounting for amount of transverse reinforcement, axial force value, concrete and steel strengths. No conventional safety factors have been accounted for. Further, mean values of material strengths have been considered, thus $f_{ck}$ has been assumed equal to $f_c=21$ MPa (achieved on cube specimens purposely tested before joint test), as well as $f_{ctm}=2.28$ MPa. The tension value exhibited by reinforcing bars has been determined for each tested specimen imposing the equilibrium of the sub-assemblage under the maximum applied horizontal force. Therefore, for each joint a different value of steel strength has been determined according with the variability of the material characteristics and the entity of the slippage effects as reported in table 5.2. In table 5.1 the failure mode is indicated as “B” in the cases in which the specimen showed a flexural failure in the beam while “J” indicates the occurrence of a diagonal cracking in the joint panel. As it can be seen, joint cracking is always accompanied by flexural cracking and yielding of beam longitudinal reinforcement.

<table>
<thead>
<tr>
<th>Test #</th>
<th>Design type</th>
<th>Axial load</th>
<th>Failure mode</th>
<th>Maximum column shear (kN)</th>
<th>Collapse drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>NE</td>
<td>NL</td>
<td>B</td>
<td>18.9</td>
<td>2.75</td>
</tr>
<tr>
<td>T2</td>
<td>Z2</td>
<td>NH</td>
<td>B</td>
<td>40.2</td>
<td>3.36</td>
</tr>
<tr>
<td>T3</td>
<td>Z2</td>
<td>NH</td>
<td>B</td>
<td>38.9</td>
<td>4.96</td>
</tr>
<tr>
<td>T4</td>
<td>Z4</td>
<td>NH</td>
<td>B</td>
<td>42.9</td>
<td>3.45</td>
</tr>
<tr>
<td>T5</td>
<td>Z2</td>
<td>NH</td>
<td>J+B</td>
<td>39.8</td>
<td>3.25</td>
</tr>
<tr>
<td>T6</td>
<td>NE</td>
<td>NH</td>
<td>B</td>
<td>21.3</td>
<td>2.85</td>
</tr>
<tr>
<td>T7</td>
<td>NE</td>
<td>NL</td>
<td>B</td>
<td>21.3</td>
<td>3.28</td>
</tr>
<tr>
<td>T8</td>
<td>Z4</td>
<td>NH</td>
<td>B</td>
<td>42.8</td>
<td>3.40</td>
</tr>
<tr>
<td>T9*</td>
<td>Z2</td>
<td>NH</td>
<td>J+B</td>
<td>48.3</td>
<td>3.30</td>
</tr>
<tr>
<td>T10*</td>
<td>Z2</td>
<td>NL</td>
<td>J+B</td>
<td>48.9</td>
<td>3.65</td>
</tr>
</tbody>
</table>

*specimens with reinforcing bars having higher strength and lower deformation capacity
In figure 5.1, as an example, the force-drift relationships and the damage pattern occurred to the specimens during the tests T3 and T5 are shown. The specimen of test T3 shows no cracks in the joint panel and all the damage is concentrated at the beam-column interface with a depth flexural crack. This joint collapsed because of the tensile failure of the bottom longitudinal bars in the beam.

The specimen of the test T5, identical to the previous one, showed a heavy damage into the joint panel in addition to the flexural cracks in the beam. The different behaviour is attributable to the value of the axial force acting in the column that was lower than in the test T5. The occurrence of the joint failure caused less satisfactory performance as displayed in figure 5.1, showing stiffness and strength deterioration as well as pinching of hysteresis loops more pronounced than in test T5.

Results of the application of code formulations reported at the paragraph 3 for estimating the joint shear capacity, are shown in figure 5.2. Design value strength of steel bars are evaluated assuming $\gamma_f$ and $\gamma_Rd$ equal to 1.0.

For each test, the first (blue) bar is relevant to the joint shear $V_{\text{exp}}$ experimentally determined, that is by using expression (3.1) as already explained at par. 4.1. For each of 10 tests both compression strength $V_{jc}$ and tensile strength $V_j$ have been evaluated through the code expressions (3.3) and (3.4), and, only for NE joints, also applying IC code expressions (3.5) and (3.6) for existing buildings.

Results in Figure 5.2a show that the shear strengths provided by code expressions are greater than $V_{\text{exp}}$, except for the tests T5, T9 and T10. In particular, tests T5 and T10 show a predicted tensile strength $V_{jt}$ lower than the experimental shear $V_{\text{exp}}$ according to the observed failure modality of the joints that involved both the beam and the joint panel (see table 5.1). As for test T9, it can be observed that the predicted compressive strength $V_{jc}$ is lower than the experimental joint shear $V_{\text{exp}}$, highlighting the occurrence of a compression failure of the joint panel. The EC8 expressions are, in these cases, able to predict the joint shear failure.
The diagram in figure 5.2b shows also the values of the joint shear strength obtained applying expressions (3.5) and (3.6) provided in IC to verify joints without transverse reinforcement belonging to existing buildings, thus applicable only to NE joints. In this case $V_{jc}$ and $V_{jt}$ (green and yellow bars) result always greater than $V_{exp}$ in accordance with the experimental result, although it cannot be stated whether the expressions (3.5) and (3.6) would be able to predict joint failure.

As for the estimation of the shear demand, in table 5.2 a comparison between the shear values experienced by the joints during the tests $V_{exp}$ and the shear demand $V_{jhd}$ calculated according to the code expression (3.1), is reported. As can be seen, the “real” shear demand values are always higher than the theoretical ones: $V_{exp} > V_{jhd}$. It is worth specifying that $V_{jhd}$ has been calculated blindly applying EC8, referring to the design yield strength of reinforcing bars used to build the specimens $f_{yd} = f_{yk} / \gamma_s = 430/1.15 = 373.9$ MPa, where $f_{yk}$ is the nominal yield strength. The mean value of the ratio $V_{exp}/V_{jhd}$ is about 1.26, highlighting the need of a correction of the strength value to be used in calculating the shear demand, as already pointed out at par. 4.1, in order to avoid a remarkable underestimation of the joint shear demand. Finally, the estimated steel strength, showed in table 5.2, has a mean value 1.24 times the nominal yield stress $f_{yk}$ and 1.08 times the mean value of yield stress $f_y$ deduced by tensile tests performed on bars actually used to build the specimens.

<table>
<thead>
<tr>
<th>Test N.</th>
<th>$V_{jhd}$ [kN]</th>
<th>$V_{exp}$ [kN]</th>
<th>Estimated steel strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>82.6</td>
<td>103.8</td>
<td>542.4</td>
</tr>
<tr>
<td>T2</td>
<td>230.3</td>
<td>264.8</td>
<td>505.9</td>
</tr>
<tr>
<td>T3</td>
<td>231.7</td>
<td>282.0</td>
<td>532.1</td>
</tr>
<tr>
<td>T4</td>
<td>196.6</td>
<td>235.6</td>
<td>521.6</td>
</tr>
<tr>
<td>T5</td>
<td>230.7</td>
<td>267.9</td>
<td>510.4</td>
</tr>
<tr>
<td>T6</td>
<td>80.1</td>
<td>103.3</td>
<td>551.1</td>
</tr>
<tr>
<td>T7</td>
<td>80.1</td>
<td>104.6</td>
<td>557.0</td>
</tr>
<tr>
<td>T8</td>
<td>196.7</td>
<td>248.5</td>
<td>545.6</td>
</tr>
<tr>
<td>T9</td>
<td>222.2</td>
<td>303.2</td>
<td>582.9</td>
</tr>
<tr>
<td>T10</td>
<td>221.7</td>
<td>322.5</td>
<td>615.9</td>
</tr>
</tbody>
</table>

Table 5.2. Comparison between experimental and code shear in joints tested at UniBas.
6 FINAL REMARKS AND IMPROVEMENT PROPOSALS

The main object of the present paper was to investigate on the experimental behaviour of RC beam-column joints, thus providing a contribution to a more reliable evaluation of the seismic vulnerability of RC existing buildings. In particular the understanding and the validation of capacity models reported in the current seismic codes is of great interest.

To this purpose, a wide bibliographic research on available capacity models and experimental investigations on beam-column joints have been firstly carried out. Literature analysis has been devoted to carefully describe some capacity evaluation models and to compare their prediction ability applying them to a database of experimental results. The capacity model of Vollum & Newmann (1999) resulted more effective in predicting shear capacity of the analysed beam-column joints. This model could be considered to improve the code expressions on joint shear strength, although further work should be made in order to enlarge the database of analysed experimental results.

The validation of the code expressions (reported in EC8 and IC) has been made on the basis of the ReLUUIS project results. The main result is that the shear demand computed by using code expression (3.1) can be unconservative if a suitable value of the steel strength is not assumed. Experimental programs performed by University of Basilicata and Udine RU's demonstrated that the actual shear demand in joints is always greater than the theoretical one, being the difference dependent on the steel type and bond conditions. Specifically, for existing buildings the underestimation of shear demand can be greater with respect to joints belonging to new buildings because, strictly applying the current code provisions, the confidence factor (CF) should divide the estimated steel strength. A more suitable application of code expressions is proposed in the present paper where mean values of material properties are multiplied by CF, as typically suggested when strength values need to be used in calculating action effects delivered to brittle component/mechanism by ductile components.

Further, the actual stress values exhibited by the steel during the tests were calculated and used for safety verifications using code expressions (3.3) and (3.4) for joint shear capacity. As a result, the code capacity models appeared able to predict whether or not the joint cracking occurred, provided that the right steel strength values were used.

Due to the limited amount of tested specimens, a more reliable proof of the effectiveness of code expressions requires further work skillfully combining purposely designed experimental investigations, review of experimental campaigns reported in the literature, and accurate numerical simulations. Particularly, extensive experimental programs on joint specimens having different characteristics (e.g. interior or exterior, bi- or tri-dimensional, beam type, etc.) well targeted on the types representative of the Italian and European built environment, need to be performed.

7 REFERENCES


AIJ - Architectural Institute of Japan (1999), Design guideline for earthquake resistant reinforced concrete buildings based on inelastic displacement concept, Tokyo, Japan.


Shin M., LaFave J. M. (2004). *Modelling of cyclic joint shear deformation contributions in RC beam-column connections to overall frame behaviour*, Structural Engineering and Mechanics; Vol 18, No.5 645-669
