A model for beam-column corner joints of existing r.c. frame under cyclic loading

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ABSTRACT: Beam-to-column joints are commonly considered critic regions for r.c. frames subjected to high-magnitude earthquakes. When designed for gravity loads only, beam-to-column corner joints can not only strongly affect the global structural behaviour of a frame, but also be a cause of its collapse. In the paper a component-based f.e. model of external beam-to-column joints is presented. The joint deformation is modelled by means of two unrelated contributions, the shear deformation of the panel zone, and the rotation at the interface sections between the joint and the structural members, due to the reinforcing bars'slip within the joint core. The first part of this work focuses on the evaluation of the joint strength and stiffness. Finally, the component-based f.e. model is validated by experimental results of tests on beam-column corner joint realized according the construction practice of the 60's-70's, confirming the effectiveness of the presented model for the assessment of existing structures.

1 INTRODUCTION

According to current seismic codes (D.M. 14/01/2008, EN 1998-1 2005), new construction design is based on the capacity design, achieving a ductile structural behaviour. However, in Italy, a large number of existing r.c. buildings are designed for gravity loads only. Past earthquake effects have shown that, often, brittle mechanisms (thus more dangerous) cause severe damage or even the building collapse. The beam-to-column joint failure, due to the exceeding of the joint shear strength or the bar bond slip, is comprised among these mechanisms.

The test carried out by Calvi et al. (2001) on a r.c. frame, designed with typical details of Italian construction practice in the '60s-'70s, showed a significant damage in the exterior joints between the first and second floor and the development of plastic hinges at the base of the columns at the ground floor. This behaviour has been also confirmed during the recent L'Aquila earthquake (2008). The development of a failure mechanism markedly different from that provided in the case of a rigid joint behaviour, for which a soft floor mechanism would be expected, was evident.

Despite the experimental evidence, typically in non-linear static and dynamic analyses, the deformability of the beam-column joints is commonly neglected: the nodal panel is assumed infinitely rigid and a verification of the strength of the joint is made only a posteriori (Lima et al. 2010). In the last two decades different f.e. models have been proposed in order to evaluate the behavior of beam-column joints subjected to cyclic loads (Alath and Kunnath 1995; Biddah and Ghobarah 1999; Youssef and Ghobarah 2001; Pampanin et al. 2002; Lowes and Altoontash 2003; Favvata et al. 2008). Nevertheless, the complexity of these models has limited their application to the assessment of existing structures, without providing a relatively simple tool for the study of seismic vulnerability of existing buildings.

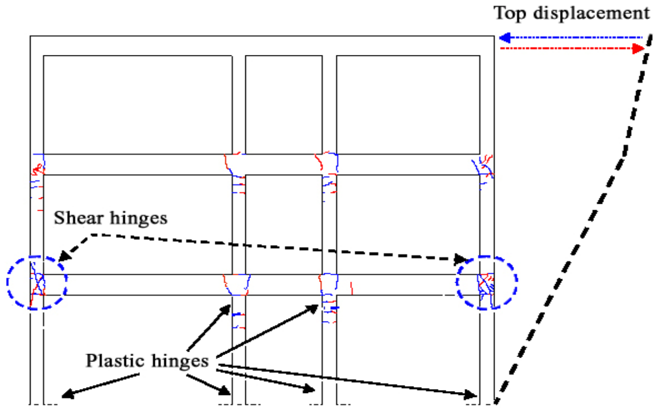

Figure 1.Global mechanism: plastic hinge and shear hinge (top drift 1.6%) (Calvi et al. 2001).

This work proposes a simple f.e. model for the nodal region of external joints in concrete frames designed for gravity loads only, focusing on details of the Italian construction practice in the '60s-'70s (hooked-end smooth bars, and no stirrups in the joint region).

The proposed component based joint model allows to evaluate separately the shear deformation of the panel zone and the added rotation at the interface sections between the joint and the structural members converging in the joint, due to the reinforcing bars' slip within the joint core. Furthermore, the model has been validated by the experimental results of tests carried out on full scale specimens, designed according to the Italian construction practice of the '60s-'70s, with respect to both material and geometrical characteristics (Beschi, 2012).

2 AN ANALYTICAL MODEL FOR CORNER BEAM-COLUMN JOINTS

2.1 *Joint shear strength*

To evaluate the joint shear strength, for exterior r.c. beam-column joint without transverse reinforcement, two alternative criteria have been considered, chosen among those available in the literature because of their simplicity and consistency.

The former, labelled in this work Principal Stress Limitation Model (PSLM), is based on the limitation of the tensile principal stress in the joint. The consistency of this model, studied specifically for joints without transverse reinforcement, with smooth hooked-ended bars, is verified in several works (Priestley 1996, Hakuto et al. 2000, Calvi et al. 2001, Masi et al. 2009).

The latter, labelled MSSTM (Modified Softened Strut-and-Tie Model), is based on the Softened Strut-and-Tie Model (SSTM) proposed by Hwang and Lee (1999) to study the behaviour of beam-column joints with transverse reinforcement. Some adjustments are herein proposed for joints designed with no transverse reinforcement and smooth bars.

2.1.1 *PSLM (Principal Stress Limitation Model)*

For joints without transverse reinforcement, a possible approach for the evaluation of the shear strength is based on the evaluation of the principal stresses in the concrete panel region (Priestley 1996; Hakuto et al. 2000; Calvi et al. 2001; Masi et al. 2009).

The principal stresses are evaluated according to the continuum mechanics equations.

The maximum tensile principal stress in the panel zone, corresponding to the development of the first diagonal crack during a first loading cycle, can be computed as follows:

$$\mathrm{p_t} = \mathrm{k_1}\sqrt{\mathrm{f_c'}} \tag{1}$$

where f_c' is the average cylindrical compressive strength of the concrete and k_1 is a constant, calibrated on experimental results, with different values proposed in the literature. In this work, the value proposed for k_1 is equal to 0.2, according to (Calvi et al. 2001).

The maximum resistant shear stress, averaged in the panel zone, is defined as:

$$v_{jh} = k_1\sqrt{f_c'}\sqrt{1+f_a/(k_1\sqrt{f_c'})} \tag{2}$$

where f_a is the mean compressive stress on the column section.

The joint panel maximum shear strength can be calculated as follows:

$$V_{jh} = v_j b_j h_j \tag{3}$$

where b_j is the effective depth of the joint and h_j is the distance between the outer column reinforcing bars.

2.1.2 *MSSTM (Modified Softened Strut and Tie Model)*

The proposed model is based on the Softened Strut-and-Tie Model (Hwang and Lee 1999), for whichthree strut-and-tie mechanisms can be recognized in the joint region with transverse reinforcement (Fig. 2(a)). The struts consist ofun-cracked compressed concrete diagonals, whereas the ties consist ofthe stirrups and the vertical column reinforcement. In the SSTM model the ultimate joint shear strength is evaluated by using an iterative procedure where at each step the stress equilibrium, the strain compatibility and the constitutive laws of materials are imposed .

The shear strength of the joint is reached whenever the compressive stress of the concrete diagonal strut is equal to the limit value given by the following expressions (Zhang and Hsu 1998):

$$\sigma_d = \sigma_{d,\lim} = \zeta\ f_c' \tag{4}$$

$$\zeta = \frac{5.8}{\sqrt{f_c'}}\frac{1}{\sqrt{1+400\ \varepsilon_r}} \le \frac{0.9}{\sqrt{1+400\ \varepsilon_r}} \tag{5}$$

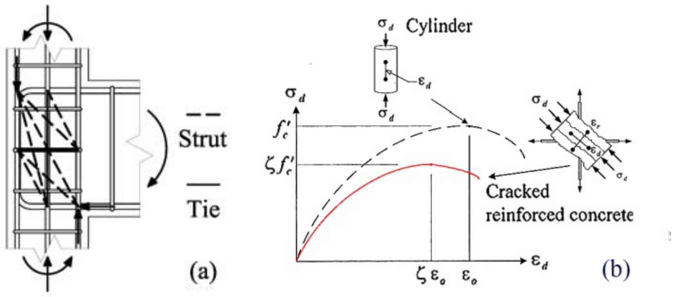

Figure 2. SSTM model: (a) struts and ties in the joint region; (b)softening of compressive stress-strain curve due to transverse tensile strain (Hwang and Lee 1999).

where σ_d is the average principal stress of concrete in the d-direction; ζ is the softening coefficient (Fig. 2(b)); f_c' is the compressive strength of a standard concrete cylinder (in units of MPa); ε_d and ε_r are the average principal strains in the d- and r-directions (r normal to d), respectively.

The iterative procedure considers the two-dimensional compatibility condition relationships, which relate the principal strains (ε_d and ε_r) with the vertical and horizontal strains (ε_v and ε_h).

With respect to an exterior r.c. beam-column joint without transverse reinforcement, the only strut-and-tie working mechanism is based on a unique diagonal concrete strut (Fig. 3).

Experimental evidence shows clearly a remarkable inclination of the strut, starting from the outside of the joint region (Fig. 3).

So, the inclination of the diagonal strut can be calculated with the following equation:

$$\theta = \arctan\left(h_p / b_p\right) \tag{6}$$

with:

$$h_p = h_b + h_s - c \tag{7}$$

$$b_p = h_c a_c / 2 \tag{8}$$

where h_b is the beam depth, h_c is the depth of the column section, h_s is the distance between the first stirrup outside the joint panel region for the column-joint interface section, c is the concrete cover, and $a_c$is the depth of the compression zone in the column, defined as:

$$a_c = \left[0.25 + 0.85 \;\; N / \left(A_g f_c'\right)\right] \; h_c \tag{9}$$

where A_g is the column cross section.
The second change to the original SSTM concerns the strut depth within the panel region, which can be defined as:

$$a_c' = a_c \sin\theta \tag{10}$$

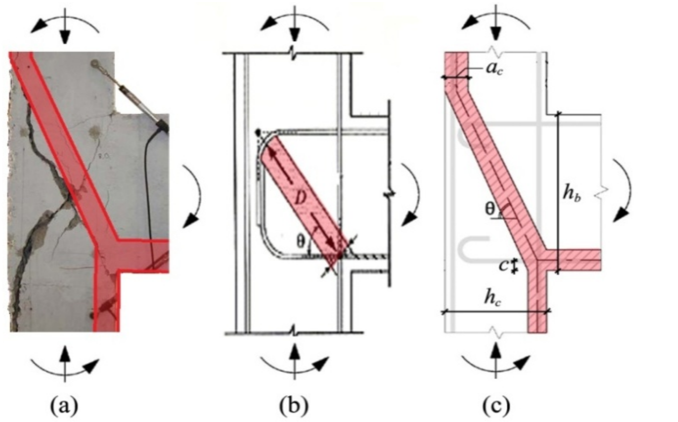

Figure 3. Inclination of the diagonal strut: (a) experimental tests (Calvi et al. 2001); (b) SSTM (Hwang and Lee, 1999); (c) MSSTM.

2.1.3 *Validations of the models*

To verify the effectiveness of the two analytical criteria, PSLM and MSSTM, with the adjustments to the original SSTM, proposed by Hwang e Lee (1999), a comparison with the results of experimental tests on joint specimens available in the literature is shown(Table 1).

All the specimens have smooth bars, unless the O7 specimen (Hakuto et al. 2000) characterized by ribbed bars bent outside the joint core. However, as specified in Pampanin et al. (2003), the failure mechanism is similar to that exhibited by joints with smooth reinforcement and hooked ends.

As shown in Table 1, the PSLM, developed to study the behaviour of beam-column joints without transverse reinforcement and smooth bars, gives a reliable estimation if compared to the experimental results, while the original SSTM, developed to study the behaviour of beam-column joints with transverse reinforcement and deformed bars, widely overestimates the experimental values, proving to be unsuitable to analyze joints with smooth bars and with no transverse reinfocements.

On the contrary, the proposed MSSTM, adaptation of the SSTM, gives a much better estimation of the experimental results.

Considering an average error at failure, the PSLM overestimates of about 5.6% the experimental strength, while MSSTM underestimates the experimental values of the same amount. It is possible to state that the adjustments made on the original SSTM allow a strut and tie method to be used also for beam-column joints without transverse reinforcement and with smooth bars.

Table 1. Average percentage error in the evaluation of the shear strength.

ERROR [%]			
JOINT	PSLM	SSTM	MSSTM
T1(Calvi et al 2001)	+11.8	+49.3	-7.4
T23-1(Braga et al. 2001)	+13.4	+27.4	-25.3
2DB(Akguzelet al., 2008)	+12.7	+67.1	+4.9
2D-B(Kam et al. 2008)	+3.9	+25.1	-24.6
O7(Hakuto et al., 2000)	-13.6	+118	+24.6
MEAN ERROR [%]	+5.6	+57.4	-5.6

2.2 *Joint stiffness*

The total deformation of a beam-column joint (Fig. 4(a)) is given by the sum of two contributions: the shear deformation of the panel zone (Fig. 4(b)) and the added rotation at the interface sections between the joint and the structural members converging in the joint, due to the slip of the reinforcing bars within the joint core (Fig. 4(c)). It is assumed that this two phenomena are unrelated, so it is possible to deal with them separately.

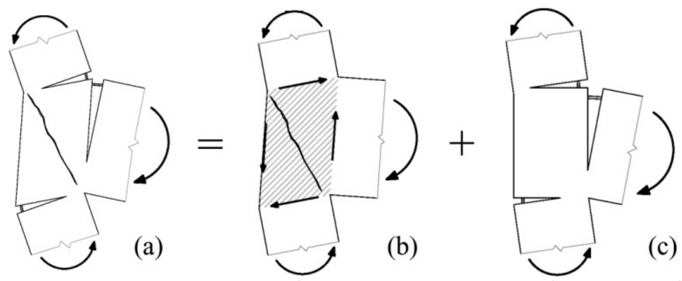

Figure 4. (a) Total joint deformation; (b) Shear deformation of the panel zone; (c) rotations at the interfaces due to bond slip.

2.2.1 *Joint panel shear deformation*

The total rotation of the joint panel due to the shear acting in the joint is given by two contributions, as shown in Figure 5:

$$\gamma_{jh} = \gamma_H + \gamma_V \tag{11}$$

with γ_H and γ_V the horizontal and vertical rotations of the joint panel sides, respectively.

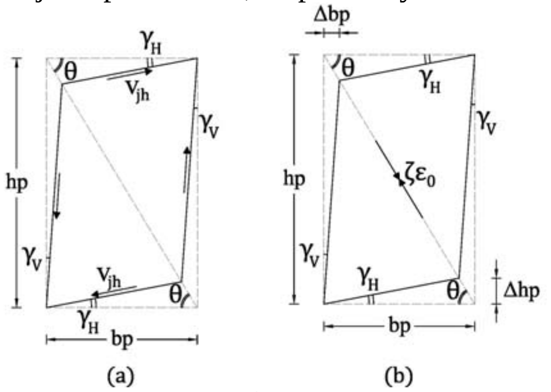

Figure 5. Shear panel deformation: (a) PSLM; (b) MSSTM.

The value of γ_{jh} depends on the method adopted for the joint shear strength calculation.By using the PSLM, the shear panel deformation can be calculated according to continuum mechanics as expressed by the following equation:

$$\gamma_{jh} = \frac{1}{G} v_{jh} = \frac{2(1+\nu)}{E} v_{jh} \tag{12}$$

where v_{jh} is the shear strength of the joint, calculated with Equation 2.

By using the MSSTM, the evaluation of the shear panel deformation is related to the ultimate strength of the compressed concrete, since the joint shear strength depends on the diagonal strut compressive strength.

At collapse, the concrete compressive strain can be defined as:

$$\varepsilon_d = \zeta \varepsilon_0 \tag{13}$$

where ε_0 is the strain corresponding to the peak compressive strength f_c' and ζ is the softening coefficient, calculated with Equation 5, used to consider the strength reduction due to the biaxial stress state in cracked concrete (Fig. 2(b)).

If ignoring the expansion of the joint panel normally to the diagonal strut due to concrete diagonal cracking, simple trigonometric expressions give the rotation γ_{jh}:

$$\gamma_{jh} = \gamma_H + \gamma_V = \frac{1}{2} \zeta\, \varepsilon_0 (\tan\theta + 1/\tan\theta) \tag{14}$$

2.2.2 *Joint panel deformation due to bond-slip*

For the evaluation of the added rotations at the interfaces between the joint and the structural members, a slip estimation of the bar straight lengthand of the hooked-end is needed. Both the contributions are modelled as two springs in series, whose stiffness determines the rotation θ_B (Fig. 6).

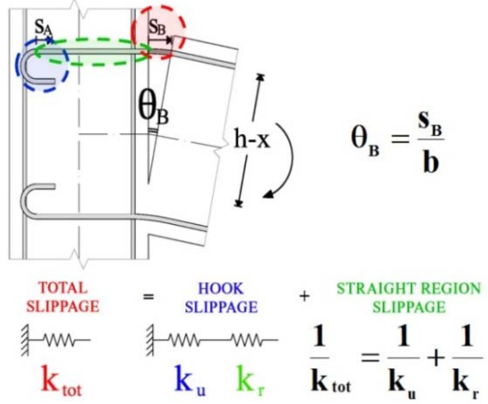

Figure 6. Contributions of bar slip to member-joint interface section rotation.

To determine the relation between the stress and the slip in the hook, the experimental curves illustrated in Fabbrocino et al. (2002) are bi-linearized using the OLS (Ordinary Least Squares) method (Fig. 7).

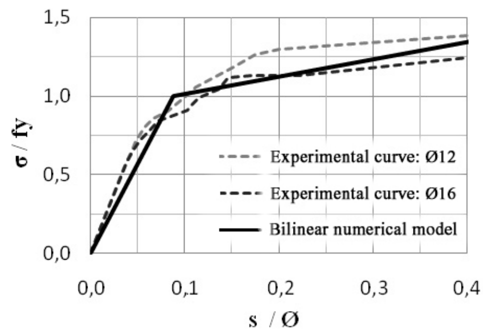

Figure 7. Stress-slip experimental curves for hooked-end bars (Fabbrocino et al., 2002).

The bond stress along the bar within the joint is supposed to be uniform whether the reinforcement remains elastic (τ_E =0.3 $f_{ck}^{0.5}$ – hot rolled bars, good bond conditions) or it is loaded beyond yield (τ_y =$\Omega\tau_E$), according to the NMC2010 draft (fib, 2010); consequently, the bar stress varies linearly. For reinforcing steel, a stress-strain relationship with hardening is adopted, with a post-elastic tangent stiffness E_h equal to the 0.5% of the elastic modulus E_s.

To evaluate the slip reinforcement along the straight region, the local equilibrium and compatibility equations are integrated along the bond length. The boundary conditions depend on whether the reinforcement remains elastic or it is loaded beyond yield, either the hook is loaded or it is not (Fig. 10). The integration allows to calculate the total slip at the interface section between the joint and the linked members.

This method can be applied to deformed bars too. Further details can be found in the report (Beschi et al. 2011). As example, Figure 8 shows the bar slip values (normalized with respect to the bar diameter) as the tensile bar stress (normalized with respect to the yielding stress) varies, and for different steel typologies.

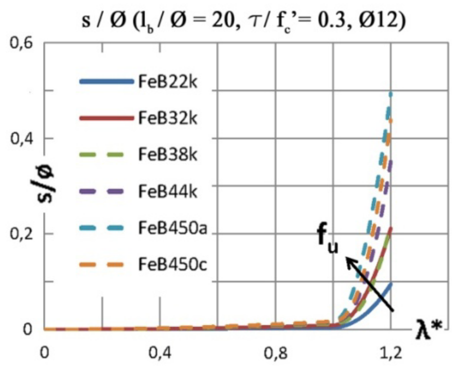

Figure 8. Normalized bar slip, with respect to the bar diameter for different steel strength.

3 COMPONENT-BASED JOINT MODEL

Over the last four decades, several approaches have been proposed for modelling the seismic response of r.c. beam–column joints, given the recognized importance of joint non-linearity on the overall structure behaviour. These models are called "component-based models", because they don't represent the joint region as a continuum, but they are made ofone or more elements representing each mechanism which contributes to the joint global behaviour.

In the present research work an f.e. explicit model has been developed, being each non-linear mechanism governing the joint behaviour taken into account, such as bar slip through the joint and shear failure in the joint core. The f.e. model has been applied to an exterior beam-column joint of a frame sub-structure. This sub-structure is made of the parts of the beam and the column converging in the joint, being included among the inflection points of the structural elements belonging to a frame subjected to horizontal actions and conventionally placed at midspan of the beam and mid-height of the column.

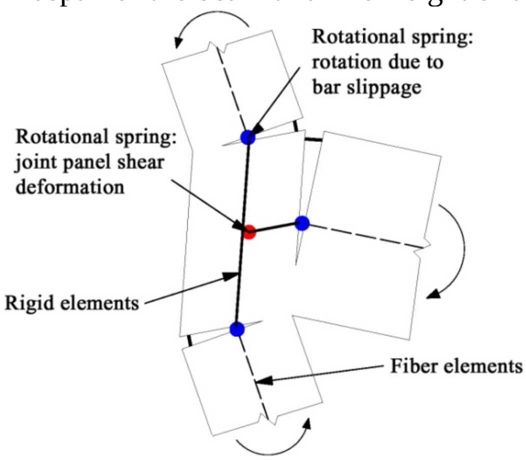

Figure 3.Joint f.e. model.

As shown in Figure 9, in the f.e. model, developed with the software MIDAS/Gen (2010), the beam and the column are modelled with fiber elements, with diffuse plasticity: the Kent & Park and the Menegotto-Pinto models have been used for the constitutive law of concrete and steel bar, respectively; the portions of the beam and the column within the joint region are modelled with rigid elements; to model the shear deformation of the joint panel region, a rotational spring is adopted, and also the deformations due to bond-slip are model with rotational springs: two springs for the column reinforcement slip and one spring for the beam bars' slip, placed at the interface between the joint and the elements converging in it.

The cyclic shear behaviour of the panel zone is modelled by a moment M_{jh} – shear deformation γ_{ih} spring adopting a hysteretic Takeda model, with a bilinear and non degrading skeleton curve to take into account the energy dissipation within the joint. The definition of the M_{jh} – γ_{ih} curve requires the evaluation of the bending moment M_{jh} acting in the beam at the column-beam intersection node and corresponding to the joint shear strength V_{ij} which generates a shear deformation γ_{ih}. The M_{jh} value is obtained by:

$$M_{jh} = \left(L_b / L_{bn}\right) \lambda \, M_{b,y} \tag{15}$$

where L_b and L_{bn} are the span between two adjacent column axes, and the relative free span of the beam, $M_{b,y}$ the beam moment resistance at the bar yielding, and λ is the fraction between the joint shear strength (V_{jh}) and the shear value which generates on the beam the bending moment $M_{b,y}$, as defined by the following equation:

$$V_{jh,y} = A_s f_y - \frac{L_b}{L_{bn}} \cdot \frac{M_{b,y}}{L_c} \tag{16}$$

where A_s is the longitudinal reinforcement cross section, and $f_y$is the bar yield strength.

Equations 15 and 16can be easily determined by applying the equilibrium equations on the modeled sub-structure (Fig. 10).

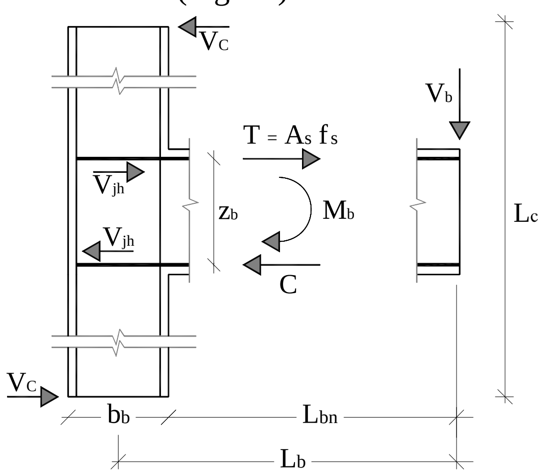

Figure 4.Forces on the modeled sub-structure.

Three rotational springs have been adopted to model the effects of the bar slip in the joint: two for column bars' slip and one for beam bars' slip, placed at the interfaces between the structural members and the joint. The model used to describe bond slip behaviour is a slip bilinear type hysteresis model, as shown in Figure 12 Thus, the first yielding moment of the section of the beam at joint interface ($M_{b,y}$), the rotation due to bond-slip of beam reinforcement when the stress in the bars is equal to the yield strength($\theta_{B,y}$), the spring elastic stiffness (k_E) and the stiffness after bar yielding (k_y) must be evaluated.

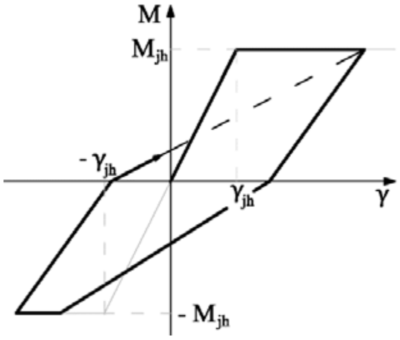

Figure 5.Takeda bilinear type hysteresis model.

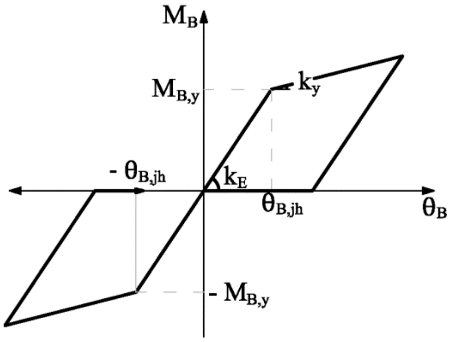

Figure 6.Slip bilinear type hysteresis model.

The rotation at beam-joint interface is supposed to be equal to the slip s_B of the rebars divided by the internal lever arm b taken equal to the difference between the beam depth (d) and the position of the neutral axis (x), conventionally considered equal to $h/3$ as shown in Figure 6.

4 VALIDATION OF THE F.E. MODEL

In this section the proposed model is applied to analyze the exterior beam-column joint, specimen T1, tested in (Calvi et al 2001) as a part of a research on the seismic behaviour of RC beam-column joints designed for gravity loads, with the typical structural deficiencies of constructions built before the ‘70s (Fig. 13).

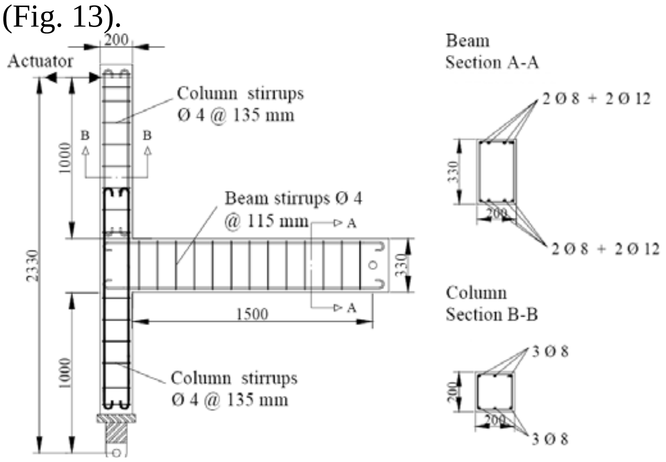

Figure 7.Geometrical characteristics and reinforcement details of specimen T1 (Calvi et al. 2001).

The mean concrete compressive strength was equal to 23.9 MPa, whereas the yielding and ultimate steel strength varied between 345 and 385 MPa, and between 403 and 458 MPa respectively, depending on the bar diameter.

Both the two criteria (PSLM and MSSTM) give a good evaluation of the joint strength, with differences in the order of 7.3% for PSLM and 11% for MSSTM, remembering that the PSLM has been developed properly for exterior beam-column joint with substandard details and calibrate on the base of the experimental results, while MSSTM is an adaptation of a model born for confined joints. Furthermore Figure 14 shows the importance of the bond-slip effects in the model to tune the numerical results to the experimental ones in term of joint stiffness and hysteretic behaviour (Tab. 2).

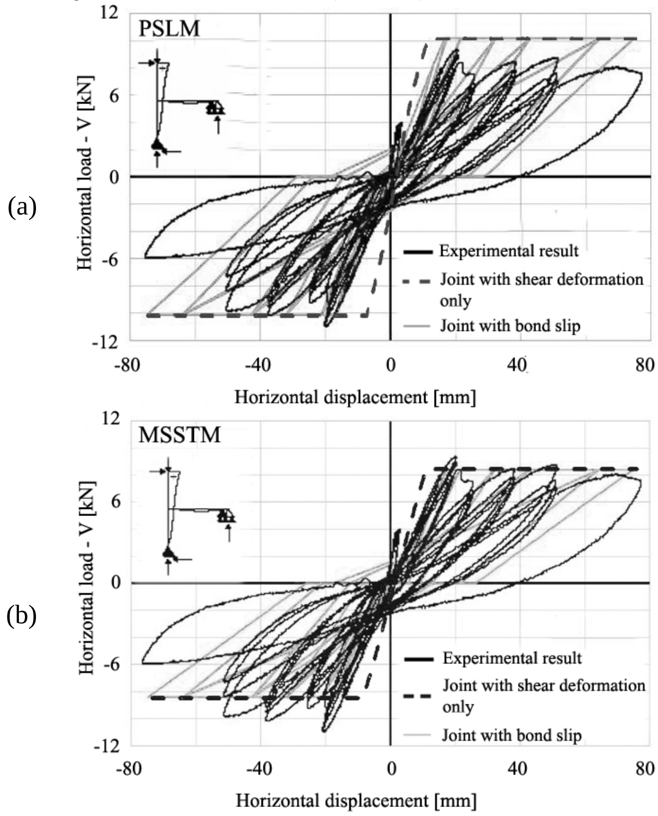

Figure 8.Comparison between experimental and numerical results on specimen T1: (a) PSLM; (b) MSSTM.

Table 2. Comparison between experimental and numerical results on specimen T1.

Specimen T1	Negative Drift V_j [kN]	K_E [kN/mm]	Positive Drift V_j [kN]	K_E [kN/mm]
EXPERIMENTAL	-10.85	0.600	8.41	0.500
NUMERICAL (PSLM)	-10.16 (-6.4%)	0.592 (-1.3%)	10.16 (+21%)	0.567 (+13%)
NUMERICAL (MSSTM)	-8.41 (-22%)	0.482 (-19%)	8.41 (0%)	0.479 (-4.2%)

V_j is the horizontal shear value which causes the joint failure.

The proposed model has been also validated by further experiment tests on full scale corner joint of the first floor of a realistic building, designed according to the Italian construction practice of the ‘60s-‘70s, with respect to both material and geometrical characteristics. The tests were carried at the University of Bergamo. A joint design was performed according to the codes provisions in force at the time of construction and the construction practice (D.M., 1974; Santarella, 1947). The specimens (labelled CJ1 and CJ2) have been designed only for gravity loads: the columns carry a centred normal action and the beams are designed according to the scheme of continuous beam on multiple supports, with upper reinforcements at the beam ends to control the crack width for service load. The column cross section is 30x30 cm, with 4 Ø16 mm longitudinal rebars and with Ø6@150 mm stirrups. The main spandrel beam is characterized by a 30x50 cm cross section,with smooth reinforcing bars with hooked-end anchorages. According to ‘60s-‘70s no transverse reinforcement were placed inside the joint (Fig. 15).Smooth steel bars, with mechanical properties similar to those typically used in the ‘60s-‘70s, have been adopted for both longitudinal and transverse reinforcement. The yield strength f_{ym} of longitudinal reinforcement was equal to 365 MPa or to 445 MPa for Ø12 mm and Ø16 mm diameter bar while it. The compressive concrete strength f_{ym} was equal to 38 MPa.

During the test, at first the column was axially loaded and a pre-load was applied to the beam simulating the gravity loads, afterwards a cyclic gradually increasing lateral displacement was applied to the column top.All the details of the test set up, test results and the calculation of all the parameters which allow to define the strength and stiffness of the springs adopted in fem can be found in (Beschi, 2012). The present paper focuses only on the validation of the proposed component-based f.e. model.

In Table 3 the comparison between the experimental and numerical results is shown: for the negative direction the specimen collapse is due to the joint shear failure while in the positive direction the failure is governed by the plastic hinge in the beam, as expected by the two analytical models.

As the two specimens showed a similar trend, the results in terms of horizontal load versus displacement curve are plotted for the specimen CJ1 only and compared to the numerical simulation (Fig.15). For negative drift both the methods are in very good agreement with the experimental values: considering the peak load value at a drift equal to -1%, the PSLM under-estimates the experimental value of about 3.8% and 2.3% for specimen CJ1 and CJ2 respectively, while the MSSTM under-estimates it of about 1.6%.

In the positive direction both methods under-estimate the experimental results with a maximum error of 13.8% (specimen CJ2).

In the numerical analyses the pinching of the hysteresis loops due to the slip of the reinforcing bars is captured, although for drift following the peak resistance, the shape of the cycles is not representative of the real behavior, as the model does not take into account the strength degradation connected to the damage in the joint panel region.

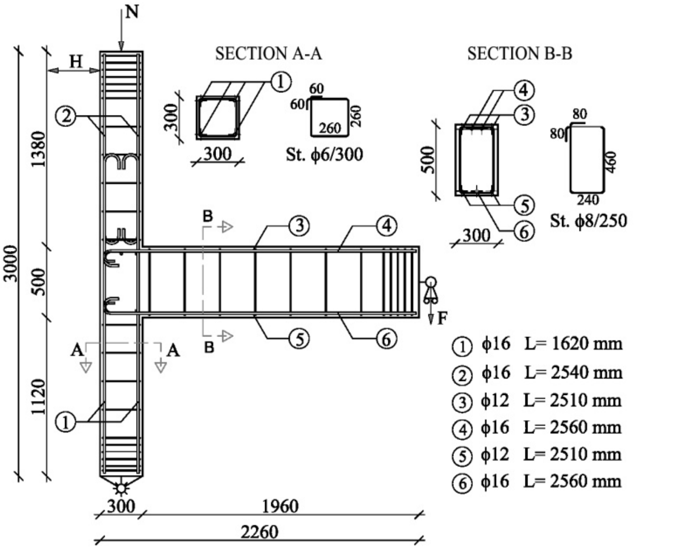

Figure 9.Geometrical characteristics and reinforcement details of tested specimen.

Table 3.Comparison of experimental and numerical results

		TEST RESULTS		Error of the numerical evaluation [%]			
				PSLM		MSSTM	
Load direction		+	-	+	-	+	-
F_{max} [kN]	CJ1	29.5	-36.0	-6.6	-3.8	-6.6	-1.6
	CJ2	32.0	-35.4	-13.8	-2.3	-13.8	0.0
K_t [kN/mm]	CJ1	1.19	-1.03	+4.6	-6.1	-9.9	+3.8
	CJ2	1.19	-1.09	+4.2	+1.0	-10.3	-1.4

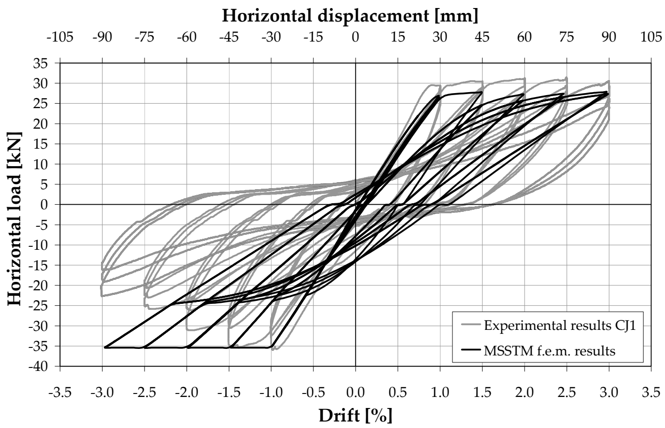

Figure 10.Comparison between numerical and experimental results for the CJ1 specimen.

5 CONCLUSIONS

Nowadays, in RC frames designed before the ’70s it is widely recognized that beam-column joints represents critical regionsunder seismic loads (in particular corner beam-column joints), influencing the response of the whole structural system, as demonstrated during recent earthquakes.

In the paper a component-based f.e. model to estimate the structural behaviour of a corner beam-to-column joint of a frame designed for gravity loads only, according to ‘60s-‘70s design practice, is presented.

The joint deformation is modelled by means of two unrelated contributions, which are the shear deformation of the panel zone, and the added rotation at the interface sections between the joint and the structural members, due to the reinforcing bars’slip within the joint core.Two analytical method are proposed to calculate the joint shear strength (PSLM and MSSTM), based on the calculation of tensile principal stress in the panel joint and of the compressive strength of the strut within the joint, respectively.

Finally, the component-based f.e. model is validated by experimental results of tests on beam-column corner joint realized according the construction practice of the 60’s-70’s, thus confirming the effectiveness of the presented model for the assessment of existing structures by the modelling of bar slip within the joint.

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