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ADVANCED NUMERICAL MODELLING FOR DAMAGE ANALYSIS OF RC STRUCTURES: A CASE STUDY ON BEAM-COLUMN JOINTS

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Abstract. *The structural damage of reinforced concrete (RC) or masonry constructions, is commonly, associated with nonlinear phenomena that make the analysis of the structural behavior a challenging task. This is even more difficult for shear critical structures. Furthermore, the severe cracking may be associated, in these cases, with an incipient collapse, due to the limited ductility of shear critical members. Advanced mechanical-based theories has been developed in order to accurately capture the post-cracking behavior of shear critical structural systems; although accurate, such theories required complex calculations that make their use not suitable for practical applications. Recent technological developments in computational capacity opened to the implementation of accurate and complex theories on the nonlinear behavior of RC structures in software suitable to accurately reproduce the nonlinear behavior of structural systems. Computer-aided nonlinear analyses are nowadays widespread in the practical engineering and, in some cases, they represent an useful tool for forensic engineering. This paper presents a case study on the application of advanced numerical modelling for the damage analysis of poorly detailed beam-column joints. These elements are one of the main source of vulnerability of RC existing buildings subjected to seismic actions and their failure may lead to the collapse of the entire structural system.*

1 INTRODUCTION

The investigation on the seismic behavior of existing structural systems is nowadays a critical issues in the protection of modern societies in seismic-prone areas. Recent earthquakes demonstrated the high vulnerability of reinforced concrete (RC) structures and in particular, those designed not conforming to current seismic codes. Concerning the beam-column joints, the 80' years large earthquakes (El Asnam, 1980, Mexico 1985, Loma Prieta 1989) clearly showed the high vulnerability of these members. Before these events the joint shear reinforcements were almost never provided. This because of the lack of widely accepted theories and formulations on the joint capacity which resulted in a complete overlook in the design and construction practice [1]. Field inspections and experimental evidence demonstrated the high seismic vulnerability of poorly detailed beam-column joints. In certain situations, the premature joint shear failure may lead to the collapse of the entire structural system. Considering that in many countries the largest development of civil infrastructures took place in the 50-80's year, it is notable that many structural systems are vulnerable to seismic action because of the reduced capacity of beam-column joint.

This strongly encouraged the scientific community to investigate the seismic behavior of joint subassemblies. Several experimental tests (an example is reported in Figure 1) pointed out the influence of different parameters (e.g. the concrete mechanical properties, joint dimensions, axial load ratio, longitudinal reinforcement type, anchorage details) on the seismic capacity of beam-column joints [1]–[4].

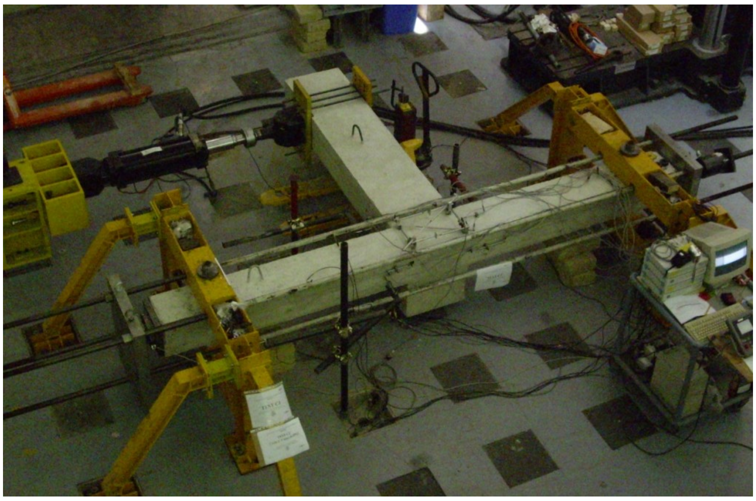


Figure 1: Experimental test on beam-column corner joint.

These observations resulted in several strength capacity models [1], [5] constituting the theoretical basis of modern code provisions for the seismic assessment of existing structures [6], [7]. Among the proposed models, particular emphasis should be given to the principal stresses approach [5]. Its mechanical basis along with the use of empirical coefficients make the model simple and suitable for practical applications. This model have been adopted in several national standards limiting the joint panel strength to the first cracking [6], [7]. Analytical studies pointed out that this limit is detrimental for the global seismic performances [8], [9]. Experimental tests [4] demonstrated that further shear forces can be carried by the joint panel after the first cracking. However, as already formulated by Priestley et al. [10] large shear deformations are exhibited after the joint panel first cracking. Indeed, to accurately reproduce the joint behavior, the joint shear deformation cannot be neglected. This is also confirmed by several research studies [2], [11] that quantified the effects of joint deformability on the overall seismic behavior.

Although several capacity model have been proposed to predict the joint behavior and the effects on the structural system performances [10]–[14], they are commonly based on empirical approaches or calibrated for specific joint types. Few models can accurately reproduce the joint hysteretic behavior. This because of the significant Strength and Stiffness Degradation (SSD) and pinching phenomena related to the shear cracking that strongly affect the cyclic response. Recent studies, pointed out the importance to accurately reproduce the member hysteretic behavior in the seismic assessment of shear critical structures [15], [16]. However include the effects of SSD in the seismic response of structural member is nowadays a critical issue. Sophisticated numerical models have been proposed to predict the structural response of SSD members for large displacement demand (i.e., the modified compression field theory, MCFT [17] and disturbed stress field model, DSFM [18]), but significant computational demand has limited their application to structures. In recent years, the development of advanced and efficient algorithms in computer programs suitable for practical applications (i.e., Membrane 2000 [19], VecTor programs [18]) promoted their use to reliably simulate the cyclic behavior of SSD RC structural systems.

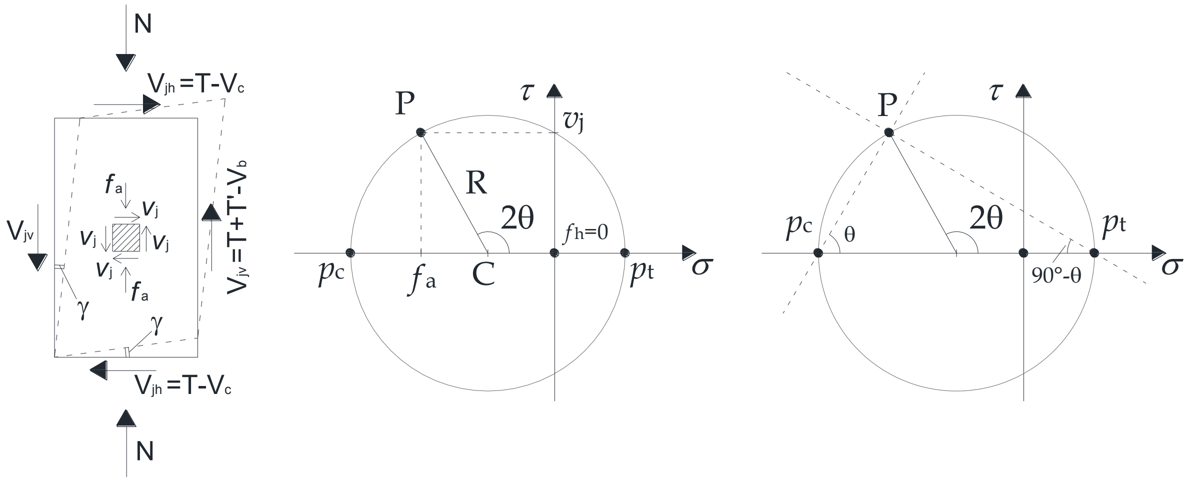
2 RESEARCH OBJECTIVE

Although the joint behavior analytical modeling assumed significant relevance in the recent years, few capacity models, reliable for poorly detailed beam-column subassembly are nowadays available. The complex hysteretic behavior of these members, characterized by significant strength and stiffness degradation and pinching phenomena related to the shear cracks, makes it difficult to be reproduced.

A new strategy to model the beam-column joint cyclic behavior, based on the recent developments in the analytical models and computer software dedicated to shear critical systems, is presented in this paper. Available finite element method (FEM)-based software, implementing rigorous theoretical approaches (i.e. the MCFT and the DSFM) have been adopted in the model development. The proposed FEM model is described in detail, with particular attention to the material mechanical models and the effects of reinforcement anchorages. The model has been validated with reference to experimental tests; in particular, comparisons between theoretical and experimental outcomes are presented and discussed in the paper in terms of global response, local stress-strain behavior and crack pattern prediction. Further considerations on the joint panel stress levels are also presented.

3 SEISMIC BEHAVIOR OF BEAM-COLUMN JOINTS

Seismic events provide reverse cyclic actions on the beam–column joints transmitted by the adjacent members. Paulay and Priestley [1] summarized the structural behavior of the joint subassemblies with simple mechanical approaches suitable for all joint types (i.e. internal, external, corner, knee). However, the strength capacity and the joint deformability strongly depend on the joint type, confinement pressure, anchorage details and longitudinal reinforcements. A reduced confinement pressure due to the lack of structural members at all the joint faces and the local actions transmitted by the beam longitudinal anchorage make the corner joint the most vulnerable against seismic actions.



(a) (b) (c)

Figure 2: Mohr's circle of stresses for a typical joint panel (a); principal stresses (b) and principal directions (b).

The joint panel is subjected to a significant and complex stress field generated by seismic excitation (bending moment, shear and axial load). The beams and columns introduce large shear forces in the concrete core. Replacing the flexural bending moments with the resulting tension, T, and compression forces, C, the joint shear force in the vertical, V_{jv}, and horizontal, V_{jh}, directions can be computed as shown in Figure 2a [1], [20]. To satisfy rotational equilibrium, the vertical joint shear must be proportional to the horizontal shear; they are approximately in proportion by the joint dimensions, h_b/h_c. These large shear forces lead to diagonal compressive and tensile stresses (Figure 2b) in the joint core that may result in severe joint cracking, especially in the case of under-designed beam–columns joints without a proper amount of internal stirrups (i.e. transverse reinforcement). A diagonal strut can be used to capture this effect. However, in the case of structural members dominated by shear, diagonal tension failure can govern over concrete strut crushing. This is particularly common on beam–column joints with a low amount of transverse reinforcement. These assumptions resulted in an analytical model based on the Mohr's circle of a typical stress field of the joint panels (Figure 2a), characterized by uniform shear stresses, $v_{jh}=V_{jh}\backslash A_{col}$, and axial stresses, $f_a=N\backslash A_{col}$. According to experimental evidence, several authors suggested to limit the average principal stresses in tension Eq. (1) or compression Eq. (2) to values proportional to the concrete compressive strength [2], [5].

$$p_t = -\frac{f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} \leq k\sqrt{f_c} \tag{1}$$

$$p_c = \frac{f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} \leq 0.5 f_c \tag{2}$$

Here, k is a numerical coefficient equal to 0.29 for deformed bars (at the joint first cracking) or 0.42 (at the peak strength). For smooth internal reinforcement the maximum peak strength is reached along with the joint cracking and k can be assumed equal to 0.20. Furthermore, with the Mohr's circle approach, the direction of the principal compressive stress, θ, can be computed; this angle represents a fulfilling approximation of the crack inclination. The experimental evidence [2], [4] showed that large diagonal corner-to-corner cracks characterize the failure mode of beam–column joint under cyclic horizontal actions, regardless of the joint dimensions or longitudinal reinforcement details. In absence of stirrups, the principal compressive stress after cracking, can be assumed to be inclined at a constant angle, θ, and, in turn, the direction of principal tensile stress is inclined at 90-θ. The angle θ can be computed, as proposed by Paulay and Priestley [1], as function of the joint panel dimensions by $\theta = atan(h_b/h_c)$.

Minor importance has been reserved to the joint deformation and its effects on the global seismic response. This because of the predominant brittle failure of these members and the difficulties in handle shear distortions. However, the joints have a key-role in the structural system and small joint shear deformations may have significant effects on the overall structural response. Although this aspect has been introduced by Priestly along with the principal stresses approach [10], only recently, the scientific community focused the attention on the role of joint shear deformations. The joint shear deformations represent the complement of the shear strength. To be more precise, they are the effects of the shear stresses applied on the joint panel (see Figure 2a). An estimation of the magnitude of the shear deformations γ, assumed equal for the horizontal and vertical direction, was introduced by Priestly [10]. Based on experimental evidence, he provided the joint panel shear deformation at different stress levels (cracking, peak strength and joint collapse). Later, this model has been particularized to include the relations for plain internal reinforcements [2].

4 ADVANCED NUMERICAL MODELLING

The advanced numerical models proposed in the recent years improved accuracy in reproducing the behavior of RC structural systems. New mechanical models and solution algorithms have been developed and great effort has been made to combine them with the state-of-the-art knowledge in refined tools suitable for the use in the practical applications (e.g., OpenSees [21], VecTor programs [22], among others).

The continuum element considered in this study is based on the Modified Compression Field Theory (MCFT) [15] and Disturbed Stress Field Model (DSFM)[18]. Both these theories have been developed at University of Toronto and validated through large experimental programs on shear critical RC panel, structural members and entire structural systems. The MCFT is a two-dimensional analytical model in which compatibility, equilibrium, and constitutive relationships were derived based on average stress and strain of concrete. In the MCFT, it is assumed that the directions of averaged stress and strain are identical. The formulation of the model was simplified by assuming cracks are smeared and fully rotating. Local stresses and strains at cracks are computed based on the stress-strain relationship developed for cracked concrete. Out-of-plane and in-plane transverse reinforcements are smeared over the entire concrete core. Such a theory has been later generalized by the DSFM, modifying some basic assumptions (i.e. alignment of shear stress and strain) and enlarging the applicability to more complex stress fields. The complexity of this approach and the number of information contained in the analysis strongly limited its their application in the past year. Recently, it has been implemented in accurate computer tools suitable for practical applications. In this paper, the response of a reference beam-column joint will be reproduced by mean of two different computer software (Membrane 2000, M2k [19], and VecTor2, VT2 [18]). They are characterized by increasing level of difficulty in the modeling and, in turn, higher accuracy in the response prediction.

The joint subassembly selected for the model validation is a poorly detailed corner beam-column joint typical of existing structures designed for gravity load only. The specimen (named T_C3 in the experimental program carried out at University of Naples and described in Del Vecchio et al. [4]) has no stirrups in the joint panel and geometry and internal reinforcement details typical of existing buildings. It has been tested with an imposed cyclic displacement applied at the beam tip and under constant axial load on the columns (see Figure 1). To simulate gravity loads, a preload of 19.2 kN was applied to the beam along with a constant axial load ($\nu = P/A_g f_c = 0.21$) to the column. The specimen was constrained to the strong floor by two rigid steel frames, with a steel roller placed inside the lower column end to simulate a pin connection. The top column was constrained to a rigid frame by two steel rollers that grabbed the column end externally and allowed top column elongation. The beam and column length were designed to allow for the typical story height and the portion of the beam up to the zero point of the bending moment diagram, respectively. The cylindrical concrete compressive strength is about 16.4 MPa and the longitudinal steel reinforcement yielding stress about 470 MPa. The experimental test pointed out a shear failure of the joint panel before the flexural yielding of longitudinal steel reinforcement of members framing into the joint.

- *Membrane 2000*

Membrane 2000 (M2k) is a simple computer program implementing the MCFT. It allows analysis of reinforced concrete shells subjected to in-plane forces (axial force in X and Y directions and in-plane shear). Internal reinforcement may be in orthogonal directions X and Y with an arbitrary number of bar layers and spacing allowed. Membrane elements subjected to in-plane forces can be found in structural walls, the webs of beams, containment vessels, and cooling towers among many others. This is the type of experimental element tested to develop the modified compression field theory. The complex stress field and geometry of a joint subassembly can be reduced to a simple shell loaded in shear and axial force by using the assumptions reported in the previous paragraph. In particular, the actions transmitted by the member framing in the joint should be reduced to acting shear stress (assumed equal for all the faces of the joint) and axial compression stresses acting on the joint horizontal faces (see Figure 2a). It should be noted that this modeling approach can be extended only to joint subassemblies suffering the premature shear failure of the joint panel without relevant nonlinear phenomena in the adjacent members. According to these assumptions, the selected joint T_C3 has been modeled in M2k by using a shell element 300 mm thick subjected to a constant axial stress of 3.50 MPa (it is obtained spreading the axial load on the joint horizontal surface). The specimen shear strength has been evaluated under monotonic joint shear stresses and the results are plotted in

Figure 3 against the joint shear strain, γ_{xy}. Comparison with available experimental results [4], points out a significant underestimation of the real joint performances in terms of joint panel peak strength (v_{jh}=2.56MPa). The analysis of the Mohr's circle of stress and the crack pattern at the peak strength (

Figure 3b,c) demonstrated that the ultimate joint capacity is reached along with joint panel first cracking, for a principal tensile stress about 1.38MPa. This is in compliance with the stress limit proposed by Priestley [5], $p_t = 0.29\sqrt{f_c}$. Such underestimation is related to the effects of the longitudinal reinforcement anchorages, neglected in the model. In fact, larger shear stress can be achieved in the joint panel if the longitudinal beam reinforcements are bent into the joint [5] ($p_t = 0.42\sqrt{f_c}$).

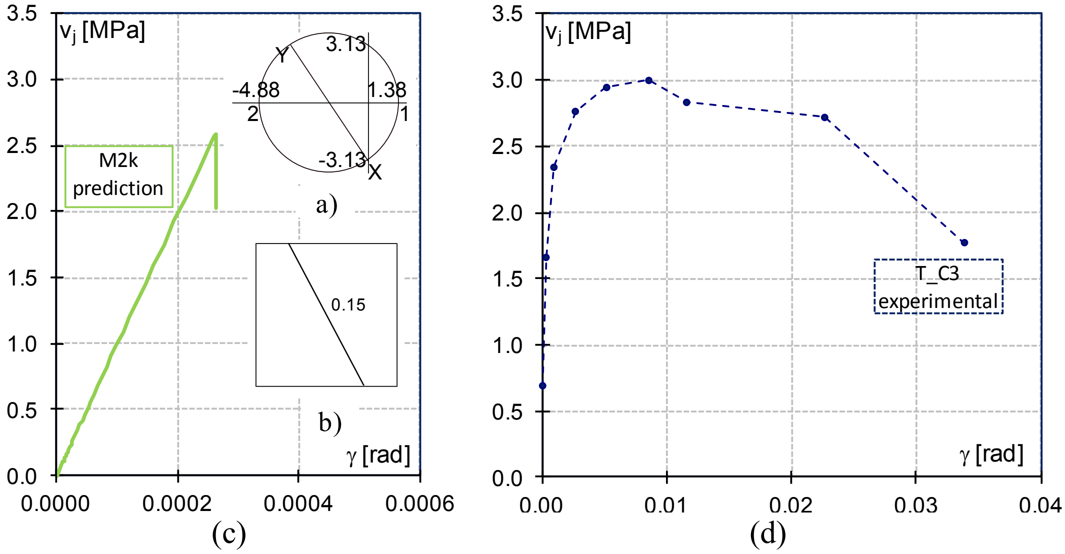


Figure 3: Membrane 2000 (M2k) monotonic response prediction without the anchorage effects: (a) Mohr's circle and (b) crack patter at the peak strength; (c) shear stress-strain behavior.

However there are no simple options to include the effects of beam bar anchorages in the M2k model. To solve this issue, the maximum lateral pressure provided by the bar bents have been computed and an equivalent amount of transverse reinforcement offering the same lateral pressure has been inserted in the model. Details on the modeling procedure are reported in Figure 4.

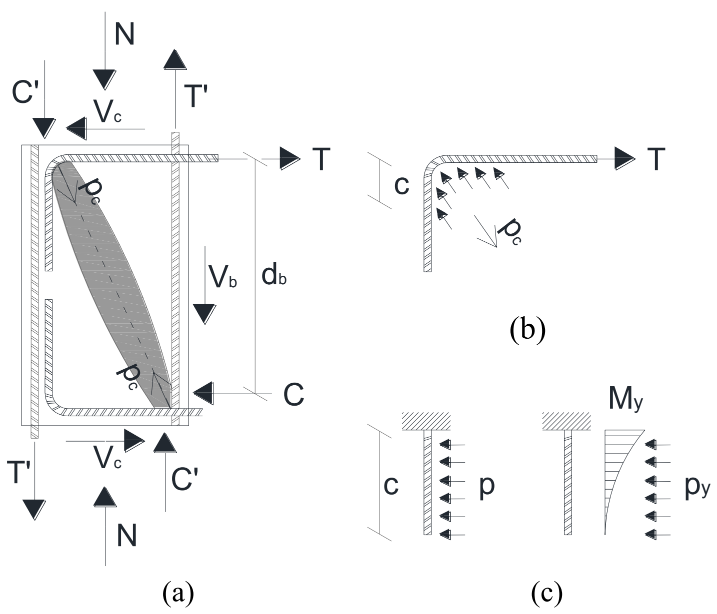


Figure 4: Mechanical model of the anchorage effects.

The lateral pressure of the beam bar anchorages comes from the reaction to the concrete compressive forces developing in the diagonal strut. The maximum pressure carried by the anchorage is assumed equal to the pressure needed to yield the longitudinal reinforcement in flexure. In the calculation it is assumed that the straight length of the anchorage is fixed in correspondence of the bent. This pressure can be computed with Eq. (3).

$$M_y = \frac{pl^2}{2} \Rightarrow p_y = \frac{2M_y}{c^2} = \frac{2f_{ym} \cdot \pi \cdot r^3}{c^2 \cdot 4} = \frac{2 \cdot 470 \cdot \pi \cdot 8^3}{85^2 \cdot 4} = 52.2\frac{N}{mm} \tag{3}$$

where c is the straight length of the anchorage subjected to the lateral pressure of the concrete compressive strut. It can be assumed equal to the total height of the concrete compressive zone above the longitudinal reinforcement $0.25h$-c_c-d_b=125-25-16=85mm, where c_c is the concrete cove and d_b is the diameter of longitudinal reinforcements.

The total pressure acting on the external face of the joint panel can be computed multiplying the maximum pressure of the single anchorage for the number of anchorages and dividing by the joint width:

$$p_{t,tot} = \frac{p_y \cdot n_b}{b_j} = \frac{52.2 \cdot 5}{300} = 0.87\frac{N}{mm^2} \tag{4}$$

where n_b is the number of beam bars bent into the joint panel, in this case 5, and b_j the joint width assumed equal to the column width, 300mm.

Once that the maximum lateral pressure has been identified, the equivalent amount of joint reinforcement (in terms of percentage of transverse reinforcements) can be derived dividing this pressure for the yield stress of steel:

$$p_{t,tot} = f_{ym} \cdot \rho_s \Rightarrow \rho_s = \frac{p_{t,tot}}{f_{ym}} = \frac{0.87}{470} = 0.186\% \tag{5}$$

In this case an equivalent amount of joint stirrup, representative of the confinement effects of the beam bar anchorages, will be placed into the joint. This solution has been preferred with respect to a constant value of the lateral pressure in order to be representative of the variability of the confinement pressure, that increases increasing the shear stress.

Comparison between the proposed analytical model and experimental results is reported in Figure 5 in terms of joint shear stress-strain behavior, acting stress field and crack pattern.

The comparison points out the good match of the proposed analytical model in terms of joint panel shear behavior at different levels. In particular, the joint panel first cracking (where hairline cracking was detected during the test) is well predicted by the proposed analytical model in terms of shear strength and crack pattern (see Figure 5, level II). Furthermore, the joint panel circle of stresses highlights the good match in terms of principle tensile stress with the limit proposed by Priestley [5], $p_t = 0.29\sqrt{f_c}$ =1.2MPa. The analytical predictions match well the experimental results also at the peak strength (see Figure 5, level III) in terms of stress strain behavior and crack pattern, where large shear cracks can be detected. The good match in terms of joint panel principal tensile stress with the stress limit proposed by Priestley, $p_t = 0.42\sqrt{f_c}$ =1.7MPa, shows the reliability of the proposed model to account for the effects of anchorages bent into the joint. The full strength degradation cannot be captured by this software and more refined calculations are needed.

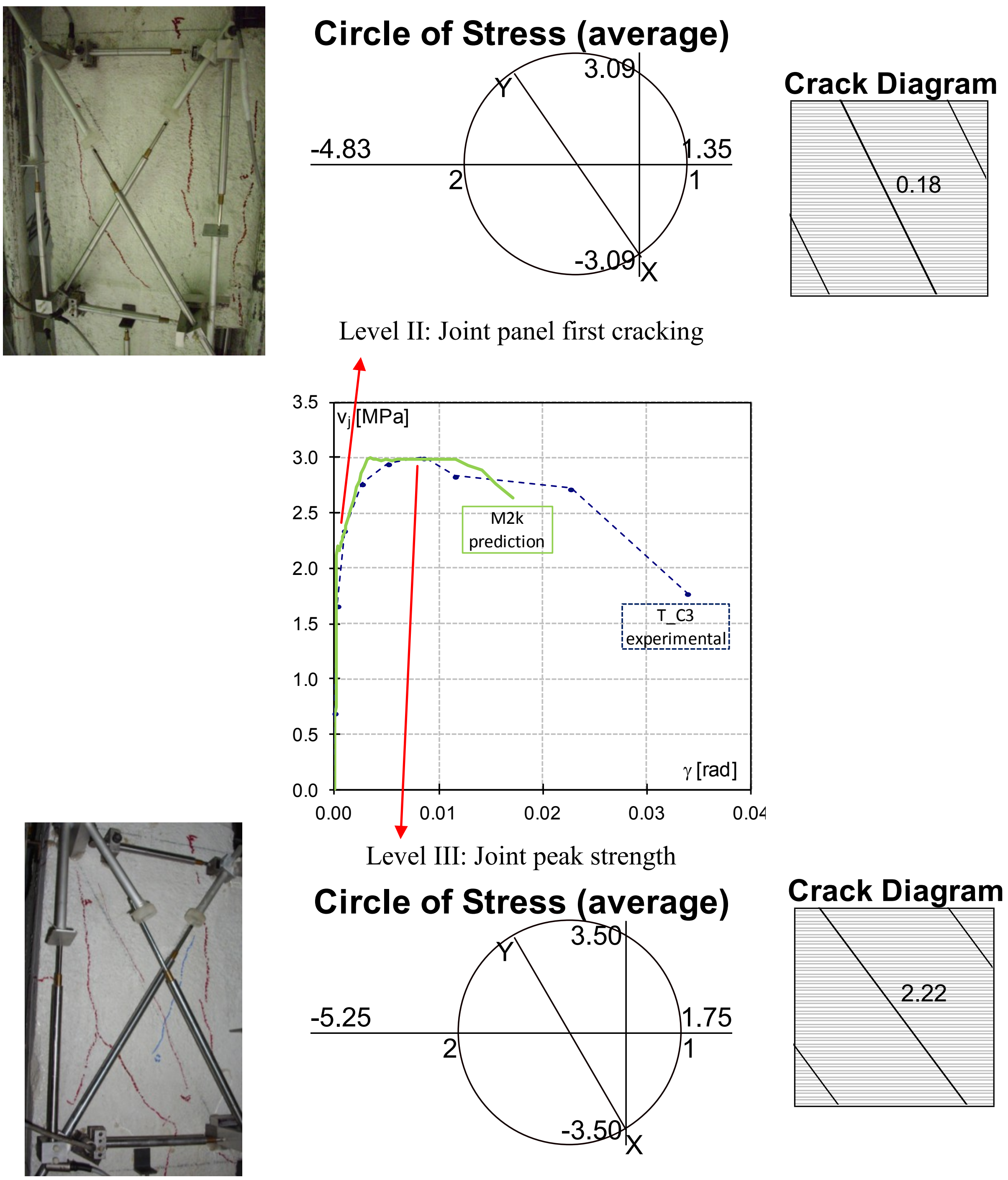


Figure 5: Experimental vs. M2k prediction in terms of shear stress-strain behavior and crack patterns (Crack widths in mm).

- *VecTor2*

VecTor2 is a program based on the MCFT/DSFM for nonlinear finite element (FEM) analysis of reinforced concrete membrane 2D structures that permits accurate assessments of structural performance (strength, post-peak behavior, failure mode, deflections and cracking). The Vec-tor2 bundle [22] includes: FormWorks, a graphics-based preprocessor program that simplifies the model building; Augustus, a complete VecTor2 post-processor that may pro-vide all the global and local results in useful numeric or graphic formats. It is also able to dis-play the specimen crack pattern at each stage of imposed displacement and this represents a very useful tool to detect numerical model failure mode. The FEM elements require a different and more complex modeling approach. The VecTor2 model was developed using a pre-processor unit FormWorks [22] that simplified the meshing and the input of the model parameters. Similar to other continuum elements, mesh size plays an important role in computational efficiency and accuracy. A mesh size in the range of about 25 mm, and approximately square elements have been adopted as suggested in the related studies [18, 25, 27]. The software can accommodate only 2D elements. A specific thicknesses equal to 300 mm has been set for all members. Longitudinal reinforcements are modeled with truss elements. Link elements were adopted to model the bond-slip behavior using the *Embedded deformed bar* option available in the software option [22]. To account for the stress concentration at the anchorages of longitudinal beam bars, the *Hooked bar option* has been adopted for the link element in correspondence of reinforcements ends. Transverse reinforcements in the beam and columns were modeled as smeared reinforcements with appropriate in-plane (ρ_t) and out of- plane (ρ_z) average ratios. The same approach was used to account for the joint panel internal reinforcement representative of the anchorage effects as determined by Eq. (5).

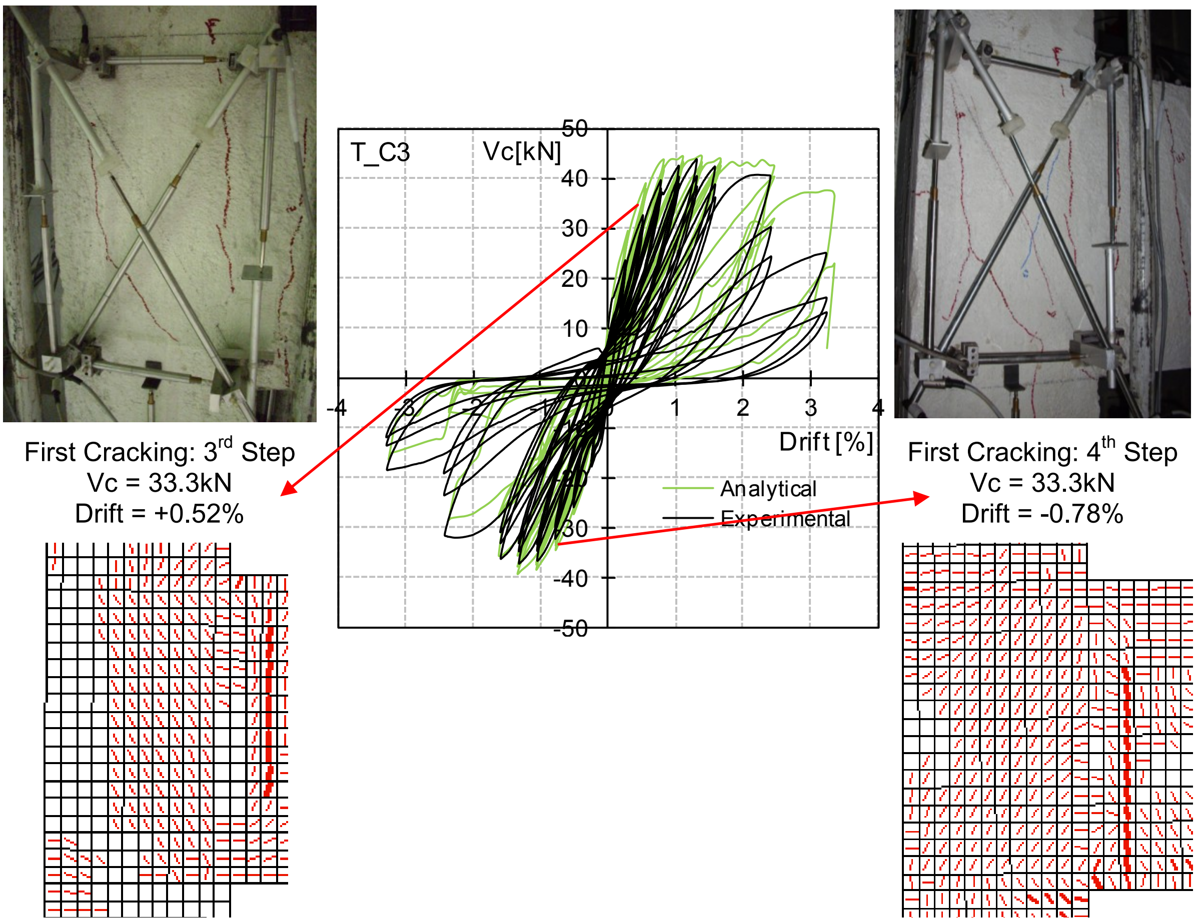


Figure 6: Experimental and analytical crack patterns at the joint panel first cracking
(Crack widths: thin<0.5mm; thick>1mm).

The concrete cover was modeled as unconfined concrete. The material properties are defined in accordance with the material tests reported Del Vecchio et al. [4]. All analyses are performed with the use of basic default material behavior models and analysis options. The concrete constitutive model by Popovic and Mander [19, 37] is adopted to reproduce concrete compressive behavior. A proper representation of the loading/unloading behavior and the cyclic-load induced damage sustained by the concrete is critical in determining the strength and energy-dissipation capacity of the subassembly. Indeed, as suggested by Sagbas et al. [23],The hysteretic model for the concrete employed here was that proposed by Palermo and Vecchio [24].
As in the experimental test, joint models are subjected to cyclic displacement and axial load applied at the beam tip and columns, respectively. The comparisons between the analytical model and experimental results at the significant steps of the test are reported in Figures 6-9.

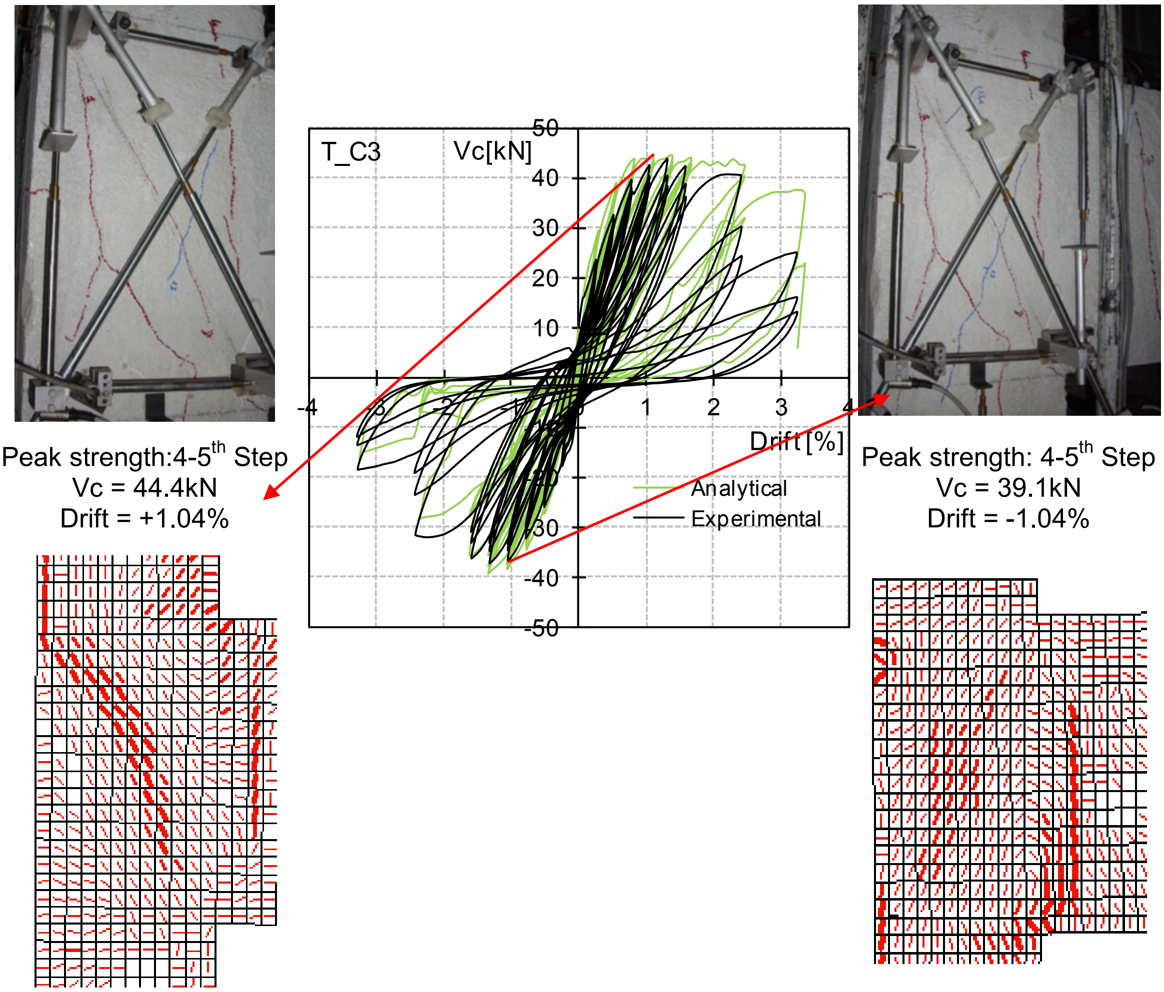


Figure 7: Experimental and analytical crack patterns at the subassembly peak strength
(Crack widths: thin<1mm; thick>2mm).

The comparison between the proposed analytical model and experimental results demonstrated the accuracy in predicting the joint subassembly global behavior. The theoretical predictions show a good match with respect to all significant parameters in the seismic assessment, including initial stiffness, peak strength, strength and stiffness degradation, pinch-

ing effects and energy dissipation. Furthermore, a crack pattern very similar to the experimental one is also predicted. In particular, the joint panel first cracking (depicted in Figure 6) occurs for a column shear of about 33kN, significantly lower than the maximum strength. At this step diffused hairline cracks appears in both directions. At the same step, a flexural crack appears on the beam in the section of the maximum bending moment. Because of the higher magnitude of the flexural crack, in the analytical model joint shear cracks are represented with a thin line.
The joint peak strength is characterized by deep and large diagonal cracks in the order of millimeters. As reported in Figure 7 a corner-to-corner diagonal crack appears in the joint panel for the positive loads. Reverse cyclic actions produces a change in the crack orientation; however, due to the strong nonlinear behavior of the cracked concrete, the opposite diagonal cracks are not completely closed.

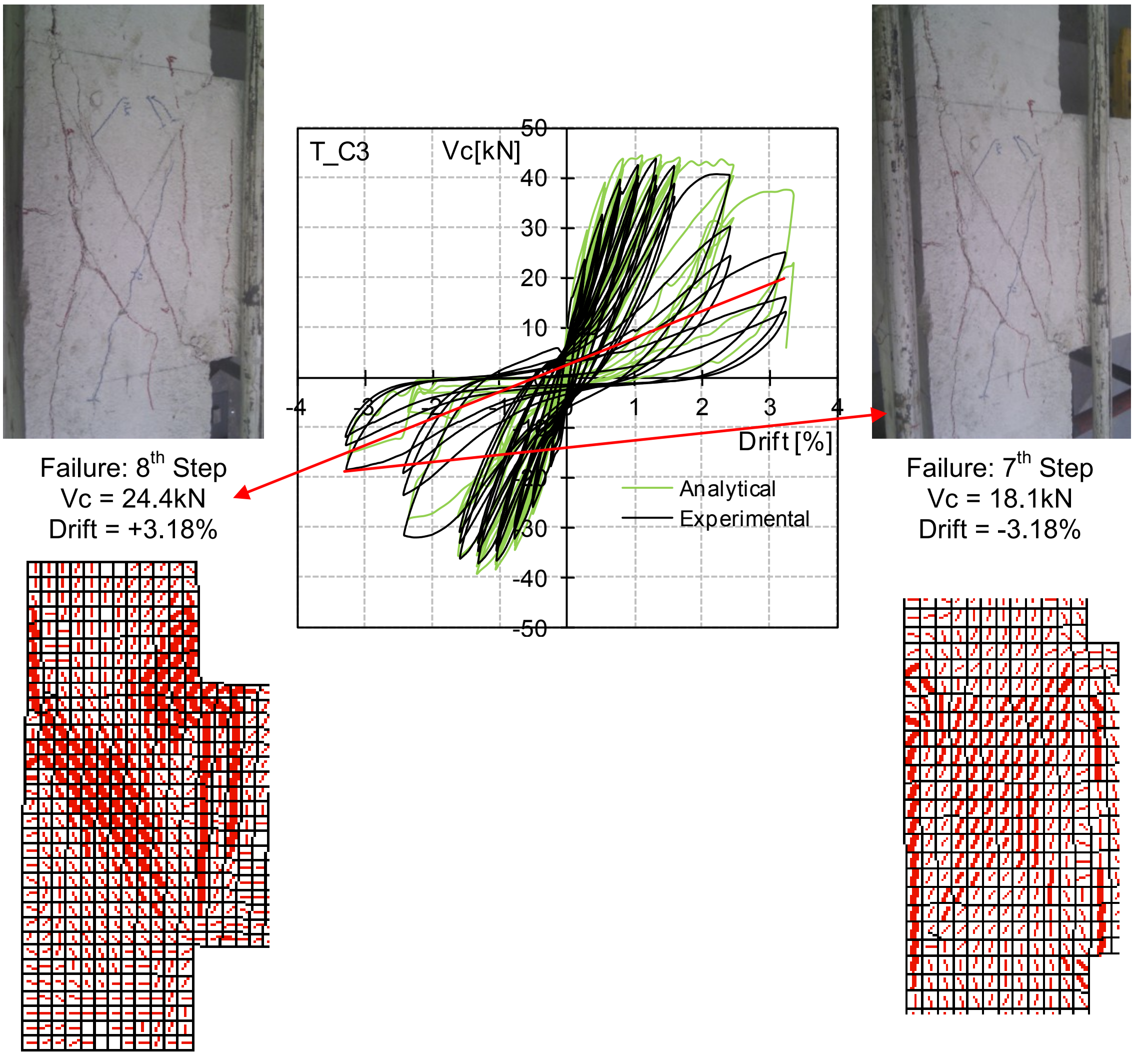


Figure 8: Experimental and analytical crack patterns at joint panel shear failure (Crack widths: thin<2mm; thick>4mm).

The crack pattern at the joint panel failure, after which a significant drop in the shear strength can be observed, shows marked cracks in both direction and the spalling of concrete cover.

Due to the severe damage of the joint panel, large shear cracks, in the order of centimeters, can be observed in both directions.

Further information on the joint panel failure mode are provided by analyzing the local behavior of members. No relevant damage is detected on columns and beam framing in the joint panel. Analytical prediction confirmed that the internal longitudinal reinforcements are far below the tensile yielding. On the other side the joint panel shear stress-strain behavior shows significant nonlinear phenomena (see Figure 9). In compliance with experimental results, the joint panel first cracking is followed by significant joint shear deformations. Once that the peak strength is achieved, the strength degradation is related to joint shear strains even double respect to the peak strength. The VecTor2 model is able to capture the shear stress-strain behavior with enough accuracy until a significant drop in the shear strength occurs.

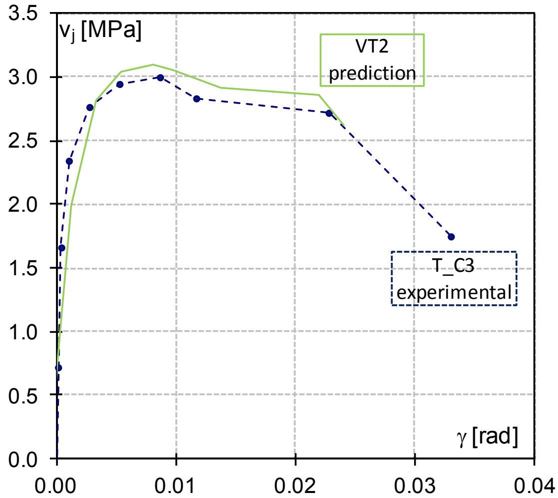


Figure 9: Joint panel shear stress strain behavior.

5 CONCLUSIONS

The seismic behavior of beam-column joint is of paramount importance in the seismic assessment of existing RC structures. In fact, the occurrence the of joint panel premature shear failure is often detrimental for the global seismic response. Several experimental tests have been carried out with the scope to characterize the joint seismic capacity and find the main parameter affecting the cyclic response. Although refined analytical models have been developed, few models are able to reliably predict the joint hysteretic response. This because of the significant strength and stiffness degradation phenomena, complex to be captured with classical modeling approaches. In the present paper, a new modeling approach for poorly detailed beam-column joints is presented. The model has been developed with the recent computer programs (Membrane 2000 and VecTor 2) able to reproduce the cyclic response of shear-critical structural systems based on refined theoretical approaches (MCFT/DSFM). The modeling assumptions on the mesh-size, mechanical model of materials, anchorage details and applied loads are discussed in detail. The comparison with experimental results of a reference tests confirms that, neglecting the confinement pressure provided by the longitudinal reinforcement anchorages in the analytical model, the strength is limited to the joint panel first cracking. The proposed M2k model, comprehensive of the anchorage effects, is able to accurately capture the joint panel shear strength at the first cracking and joint ultimate capacity.

Furthermore, the analytical response in terms of principal stress is in compliance with the limits proposed by Priestley, both at the first cracking and peak strength.

The more refined VecTor2 analytical model provide a good match with experimental results with respect to all significant parameters in the seismic assessment, including initial stiffness, peak strength, strength and stiffness degradation, pinching effects and energy dissipation. Furthermore, the crack pattern at different stress level can be accurately predicted. Concerning the local stress-strain behavior of the joint panel, a surprising match has been also achieved. In conclusion, the proposed analytical model can be employed to accurately predict the response of poorly detailed beam-column joint. However, particular care should be adopted in modeling the anchorage details to avoid significant underestimation of the joint shear capacity.

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REFERENCES

[1] T. Paulay and M. J. N. Priestley, *Seismic design of Reinforced Concrete and masonry buildings*. U.S.A.: John Wiley & Sons, Inc., 1992, p. 744.

[2] G. M. Calvi, G. Magenes, and S. Pampanin, “Relevance of beam-column joint damage and collapse in RC frame assessment,” *J. Earthq. Eng.*, vol. 6, no. Special Issue 1, pp. 75–100, 2002.

[3] A. Prota, A. Nanni, G. Manfredi, and E. Cosenza, “Selective upgrade of underdesigned Reinforced Concrete beam-column joints using carbon Fiber-Reinforced Polymers,” *ACI Struct. J.*, vol. 5, no. 101, pp. 699–707, 2004.

[4] C. Del Vecchio, M. Di Ludovico, A. Balsamo, A. Prota, G. Manfredi, and M. Dolce, “Experimental investigation of exterior RC beam-column joints retrofitted with FRP systems,” *ASCE J. Compos. Constr.*, vol. 18, no. 4, pp. 1–13, 2014.

[5] M. J. N. Priestley, “Displacement-based seismic assessment of Reinforced Concrete buildings,” *J. Earthq. Eng.*, vol. 1, no. 1, pp. 157–192, Jan. 1997.

[6] CEN, *Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and reofitting of buildings*. Brussell: European Committee for Standardization, 2005.

[7] MI, *D.M. 14 Gennaio 2008 (D.M. 2008). Technical codes for structures (in Italian)*, GU n°29, F. Rome, Italy, 2008.

[8] M. Di Ludovico, G. Manfredi, E. Mola, P. Negro, and A. Prota, “Seismic behavior of a full-scale RC structure retrofitted using GFRP laminates,” *J. Struct. Eng.*, vol. 134, no. 5, pp. 810–821, 2008.

[9] R. Frascadore, M. Di Ludovico, A. Prota, G. M. Verderame, G. Manfredi, M. Dolce, and E. Cosenza, “Local strengthening of RC structures as a strategy for seismic risk mitigation at regional scale,” *Earthq. Spectra*, vol. In Press, 2014. http://dx.doi.org/10.1193/122912EQS361M.

[10] M. J. N. Priestley, F. Seible, and G. M. Calvi, *Seismic design and retrofit of bridges*. New York: John Wiley & Sons, Inc., 1996, p. 686.

[11] S. Park and K. M. Mosalam, "Simulation of Reinforced Concrete Frames with Nonductile Beam-Column Joints," *Earthq. Spectra*, vol. 29, no. 1, pp. 233–257, Feb. 2013.
[12] L. N. Lowes and A. Altoontash, "Modeling Reinforced-Concrete Beam-Column Joints Subjected to Cyclic Loading," *J. Struct. Eng.*, vol. 129, no. 12, pp. 1686–1697, 2003.
[13] ASCE/SEI, *ASCE/SEI 41-06. Seismic rehabilitation of existing buildings*. Reston, VA, USA, 2007.
[14] J. M. Lafave and J. Kim, "Joint Shear Behavior Prediction for RC Beam-Column Connections," *Int. J. Concr. Struct. Mater.*, vol. 5, no. 1, pp. 57–64, 2011.
[15] X. Huang and O.-S. Kwon, "Numerical Models of RC Elements and their Impacts on Seismic Performance Assessment," *Earthq. Eng. Struct. Dyn.*, vol. 44, no. 2, pp. 283–298, 2015.
[16] C. Del Vecchio, O. Kwon, L. Di Sarno, and A. Prota, "Accuracy of nonlinear static procedures for the seismic assessment of shear critical structures," *Earthq. Eng. Struct. Dyn.*, vol. (on line published, 2015). doi: 10.1002/eqe.2540.
[17] F. J. Vecchio and M. P. Collins, "The Modified Compression-Field Theory for Reinforced Concrete elements subjected to shear," *ACI J.*, vol. March-Apri, pp. 219–231, 1986.
[18] F. J. Vecchio, "Disturbed Stress Field Model for Reinforced Concrete: Formulation," *ASCE J. Struct. Eng.*, vol. 126, no. 9, pp. 1070–1077, 2000.
[19] E. C. Bentz, "Sectional Analysis of Reinforced Concrete Members," University of Toronto, ON, Canada, 2000.
[20] C. Del Vecchio, M. Di Ludovico, A. Prota, and G. Manfredi, "Analytical model and design approach for FRP strengthening of non-conforming RC corner beam-column joints," *Eng. Struct.*, vol. 87, pp. 8–20, 2015.
[21] The Regents of the University of California, "OpenSees: Open system for earthquake engineering simulation." Pacific Earthquake Engineering Research Center College of Engineering University of California, Berkeley.
[22] P. S. Wong, F. J. Vecchio, and H. Trommels, "VecTor2 & FormWorks User's Manual. Second Edition," Toronto, ON, Canada, 2013.
[23] G. Sagbas, F. J. Vecchio, and C. Christopoulos, "Computational Modeling of the Seismic Performance of Beam-Column Subassemblies," *J. Earthq. Eng.*, vol. 15, no. 4, pp. 640–663, Apr. 2011.
[24] D. Palermo and F. J. Vecchio, "Compression Field Modeling of Reinforced Concrete Subjected to Reverse Loading: Formulation," *ACI Struct. J.*, vol. 100, no. 5, 2003.