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NONLINEAR SEISMIC ANALYSIS OF GRAVITY DESIGNED RC STRUCTURES

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Abstract. *This paper illustrates numerical simulations and comparisons with experimental results of existing RC beam-column sub-assemblages reinforced with smooth bars. The specimens reproduce parts of concrete structures designed only for vertical loads without any reinforcing detail rule (such as inadequate bars lap splice, absence of hoops within the joint panel) built in Italy during '50s-'70s. In particular, in this paper is paid attention to the nonlinear models developed for predicting the failure mechanism experimentally observed, taking also into account the bond-slip phenomenon among the longitudinal bars and surrounding concrete. The proposed models are not time-consuming and may be easily implemented in any general-purpose finite element program for numerical simulations of concrete structures. These models represent an useful tool for seismic assessment with a good accuracy of non-ductile RC existing structures.*

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1 INTRODUCTION

Starting from the last decades there has been an increasing attention to understand the behavior of old existing reinforced concrete (RC) structures in seismic prone areas. Most of them result still in service although they are extremely vulnerable with respect to the seismic action because of they were built only for vertical loads and poorly detailed, with a consequent limited ductility.

The observation of the damage occurred during the seismic events and laboratory tests have highlighted that the response under lateral actions of old existing RC is inadequate already for moderate seismic actions. The low-ductile behavior of these structures is mainly due to: scarcity of transverse reinforcement within the critical regions of beam and columns, absence of hoops for shear into the joint panel areas; inadequate lap splice length of longitudinal bars. These structures under seismic actions are also very often dominated by bar slippages, that reduce the lateral stiffness and the hysteretic energy dissipated.

In this paper are shown the results obtained by using numerical models for seismic assessment of old existing sub-assemblages reinforced with smooth bars [1]. The models have been developed starting from the failure mechanisms observed during an experimental campaign carried out on prototype connections designed only for vertical loads [2], and supposed to belong to typical RC buildings constructed in Italy and Mediterran area during the '50s-'70s. In the proposed models the bond-slips of longitudinal bars are incorporated at material level by applying a modified steel model developed in previous works [3, 4, 5]. At first the main results of the experimental campaign are commented. Then, the proposed models are described and applied for simulating the experimental response of the tested sub-assemblages.

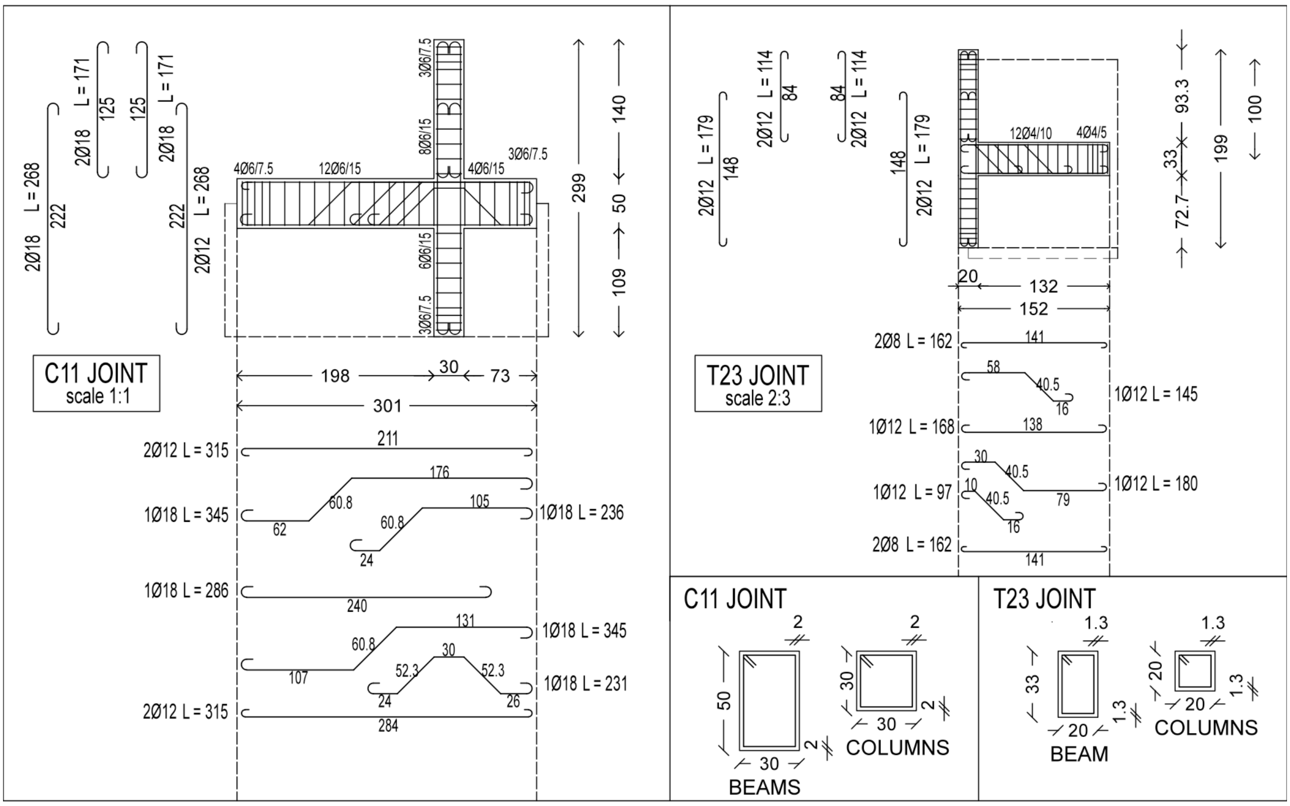

Figure 1: Reinforcements detail of C11 and T23 joints. Dimensions are in centimeters and bars diameters in millimeters.

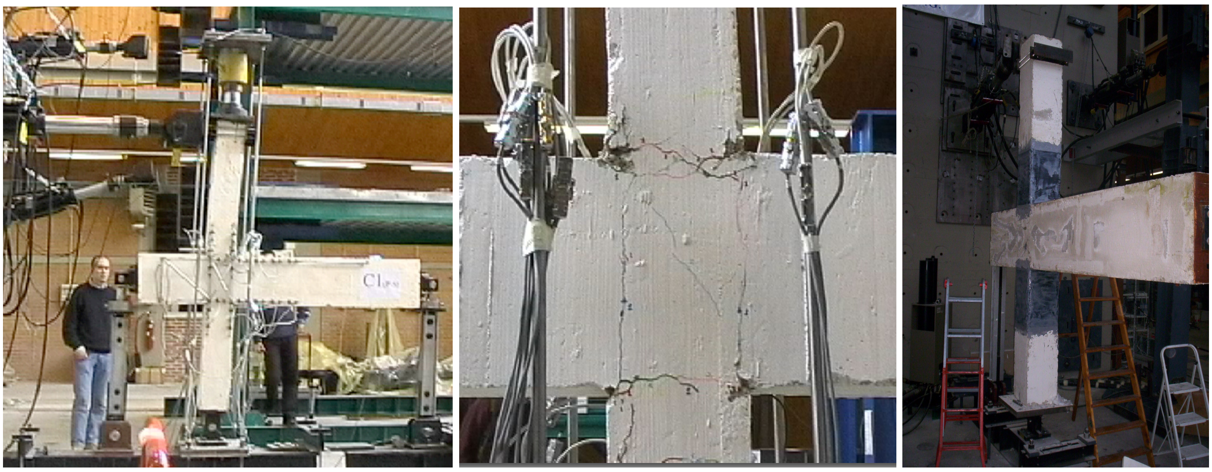

Figure 2: An image of the C11 joint during the first test (on the left), the observed crack patter (in the center) and the application of CFRP wraps before the second test (on the right).

2 EXPERIMENTAL RESULTS

An experimental campaign [2] was focused on beam-column joints reproducing internal (named "C" joints) and external ("T" joints) connections of a non-ductile prototype RC frame (Braga et al. 2009), built in Italy during the '50s-'70s and designed only for vertical loads in according to the Italian Designed Code R.D 1939 [6]. The prototype consisted of 3 floors and 3 spans, and the beam-column connections were cast in both full scale and in scale 2:3. Both the types of beam-column joints were detailed in according to the practice adopted in the past. Therefore, they were poorly detailed within the regions converging into the joints, and no transverse reinforcement was provided within the joint core. In Figure 1 are reported the details of the joint named C11 and T23, whose experimental results are discussed in this paper.

2.1 Internal joint

Three internal-beam-column joints were tested in a quasi-static-way by applying at the top of the upper column an increasing displacement history. For all the tested specimens, a strong column-weak beam response mechanism was observed already from the early drift levels. The observed mechanism may be schematically summarized as alternating rigid rotations of the upper and lower columns. It developed with limited damaging at the columns ends near the joint panel, where flexural hinging took place with pronounced bond-slips of columns passing bars. No damage was observed within the joint panel area, remaining undamaged until the specimen failure.

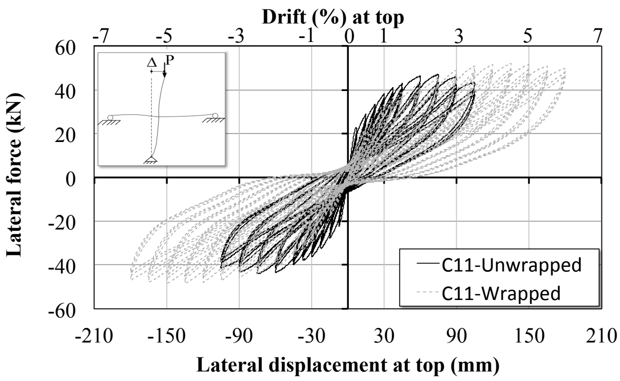

Figure 3: Joint C11. Experimental response obtained during the first (Unwrapped) and the second test (Wrapped).

Here is discussed the experimental response of the sub-assemblage named "C11-joint". This specimen was tested two times. The first test was performed up to a moderate damage level (maximum drift of 3.5%) with the scope of preserving the specimen integrity. Then, the same specimen was repaired and tested again under the same vertical load up to the failure (the maximum drift reached was of 7%). In this way, it was simulated the procedure of local repairing and strengthening of the columns ends when a moderate seismic event occurs on old existing RC structures. The local strengthening consisted of three layers of CFRP wraps applied with the scope of increasing the confinement level within the columns critical ends. In Figure 2 are reported some images of C11 tested during the first test, and after the application of CFRP wraps. More details about the tests may be found in [1]. In Figure 3 are compared the global responses obtained on the C11 during the first (C11-unwrapped curve) and the second test (C11-wrapped curve). It easy to recognize that the cycles, also at early stages, are characterized by pinched hysteretic loops due to the significant bond-slips of longitudinal passing bars. The repairing intervention provided stability to the global response, inhibiting the concrete degradation and increasing, very significantly, the specimen ductility.

2.2 External joint

In [2] was tested an external joint (named T23) cast in scale 2:3, subjected to an increasing lateral displacement at the top of the upper column. The observed failure mechanism was characterized at first by the formation of diagonal cracks within the joint panel, and then by a series of alternating rigid rotations of the two columns. The rotations arose around the idealized point A and B indicated of Figure 4. It is worth to note that the outer longitudinal passing bars were, after the diagonal crack opening, always in tension. The point A and B represent the pressure centers of the joint panel where the vertical local is transferred to the base of the lower column. Also, it must be remarked that no slippages of hooks of beam bars towards neither inside nor outside the joint panel was observed. Hence, the ejection of the formed triangular block was not influenced by any hook slip.

The experimental response of the external joint in terms of lateral force applied versus the imposed displacement at the top is depicted in Figure 5. It is easy to note that, although the response mechanism was characterized by a shear failure, its lateral response showed a limited, and not neglecting, hysteretic capacity. This was due to the significant slippages of passing bars and to a reduced deterioration of the concrete in correspondence of the rotation centers A and B.

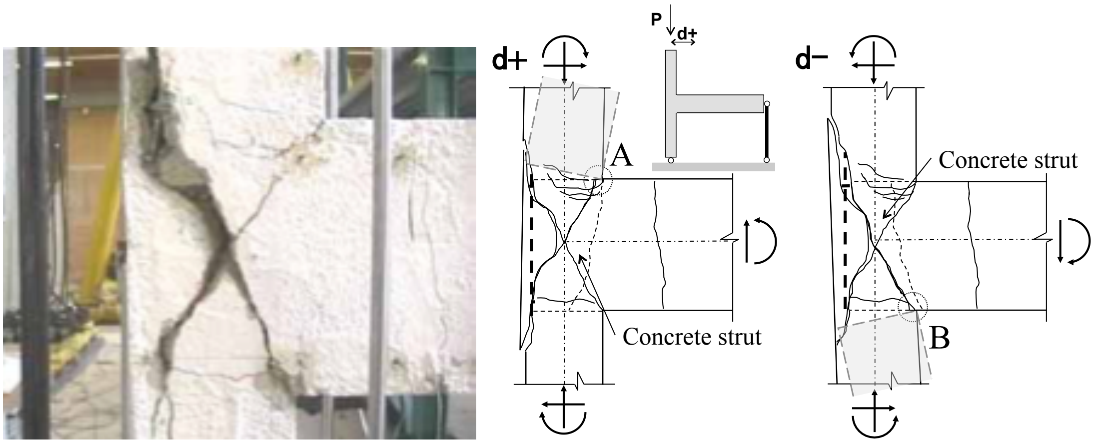

Figure 4: Joint T23. Experimental response obtained during the first (Unwrapped) and the second test (Wrapped).

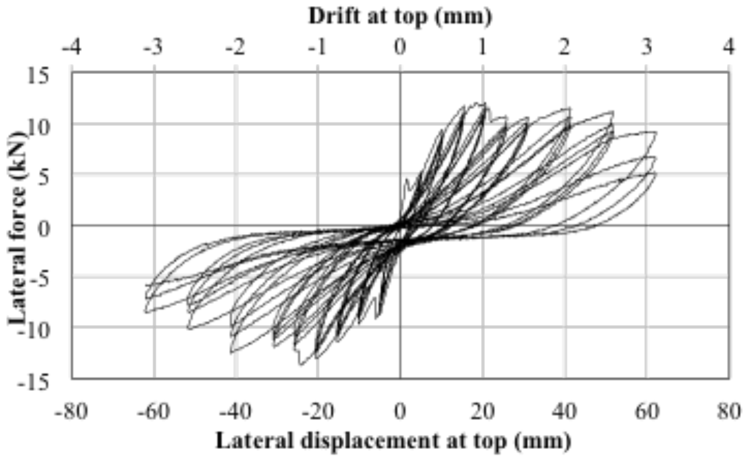

Figure 5: Experimental response of the external joint T23.

3 NUMERICAL MODELS

The numerical models proposed for predicting the internal and external beam-column joints response are hereinafter described and applied. The models are defined only on geometrical and mechanical properties of elements, without applying any empirical calibration procedure. They require of implementing elastic and nonlinear mono-dimensional elements. In this work, the proposed models are implemented in OpenSees [7], an objected open-source framework for finite element analysis developed at the PEER (Pacific earthquake Engineering Research center). More details on the proposed models may be found in [1].

In the models the slippages are incorporated within the steel constitutive law of longitudinal reinforcements, in according to the model proposed by [3] and validated in [4]. As for the effects of confinement the concrete analytical model proposed in [8] and implemented in [9] is applied.

3.1 Internal joint

The numerical model proposed for the internal joint C11 consists of four *BeamWithHinges* elements, having fiber regions at both elements ends. These elements have an elastic middle region where the uncracked elastic stiffness of concrete is assigned (Figure 6).

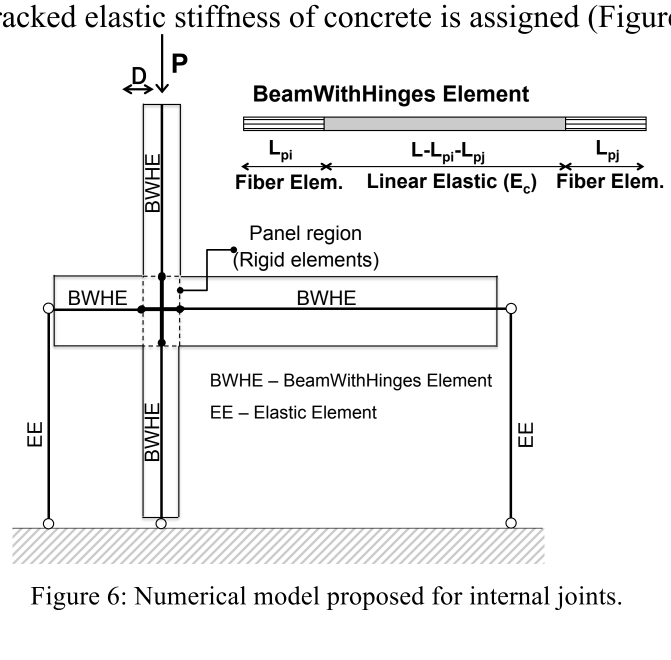

Figure 6: Numerical model proposed for internal joints.

The hinge length of elements L_{pl} has been assumed equal to *h/3*, where *h* is the section element. Rigid elements are used for modeling the joint panel, since during the experimental tests this concrete region remained undamaged until the specimens failiure. Due to the inadequate anchorage conditions within the joint panel, only tensile strength to the steel bars is assigned. This assumption takes into account the loss of anchorage of columns passing bars, and it becomes particularly appropriate at higher magnitude of the applied displacements.

3.2 External joint

The finite element model proposed for the external joint T23 is reported in Figure 7. The model is composed by *BeamWithHinges* element having, in according to the measured experimental values, a hinge length of $L_{pl}=h/3$ (*h* is the section depth of columns and beam). Hysteretic trusses are used for modeling the joint panel area, linked to a rigid elements contour connected to the converging beam and columns. The rotation center A and B of the two columns are modeled with two unit-length rigid elements, having negligible bending stiffness. The two diagonal trusses act in parallel under the imposed lateral force for reproducing the alternated opening and closure of diagonal cracks. The axial strength in tension (for simulating the crack initiation) and in compression (for simulating the development of the alternated columns rotations) of the two trusses are calculated starting from the Mohr's circles as described in [1]. The two trusses have a hysteretic behavior for realistically simulating the evolution of the response mechanism under the imposed cycled displacements.

The proposed model does not reproduce the response before the initiation of joint panel diagonal cracks, since it is only capable of simulating the alternating rigid rotations of the two columns. It does not consider the application point variation of the columns vertical loads due to the imposed displacement history. Therefore, it is only adequate to reproduce the experimental behavior of the sub-assemblage at the larger levels of the imposed displacement.

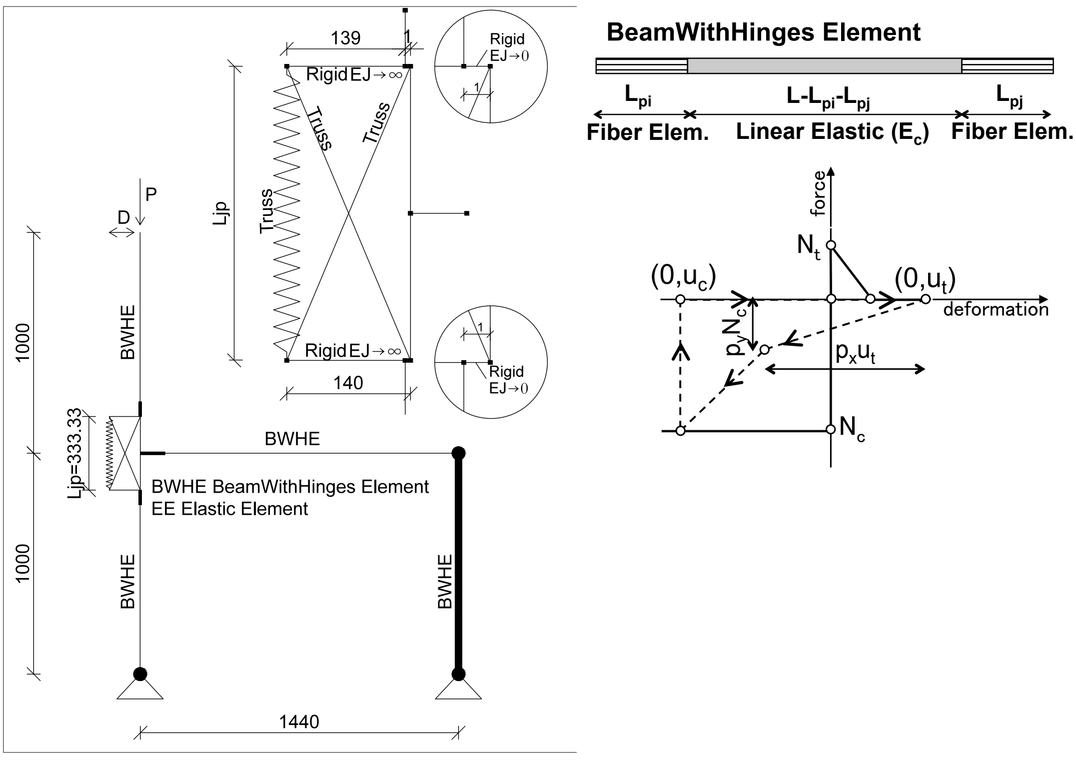

Figure 7: Numerical model proposed for external joints.

4 COMPARISONS WITH EXPERIMENTAL RESULTS

4.1 Internal joint

The comparisons among the experimental results and the ones obtained with the proposed model are reported in Figure 8 and Figure 9. For both the C11 tests, the comparisons are firstly shown in terms of envelope curves, and then in group of cycles (corresponding to the drift levels of 1%, 2% and 3%, respectively). For completeness, to the steel of longitudinal bars different constitute laws are assumed: the classic full bond assumption (full bond curve), and by taking into account the bond slips of passing bars, either in the case of full strength in compression (Bond-slips 100% curve) or in the case of loss of anchorage (no strength in compression, Bond-slips 0% curve).

The comparisons shown highlight the importance of the bond-slips and of the loss of anchorage of longitudinal bars on the global response of the sub-assemblage. When external CFRP are applied (second test of the joint C11) an important increasing of specimen ductility is reached. The proposed model matches very well the experimental response for both C11 tests (unwrapped and wrapped conditions) in terms of global strength and dissipated hysteretic energy, especially for larger values of the imposed drift. When the C11 joint is repaired, the slight difference in terms of initial stiffness (either positive or negative) is mainly due to the damage accumulated by the elements during the first test.

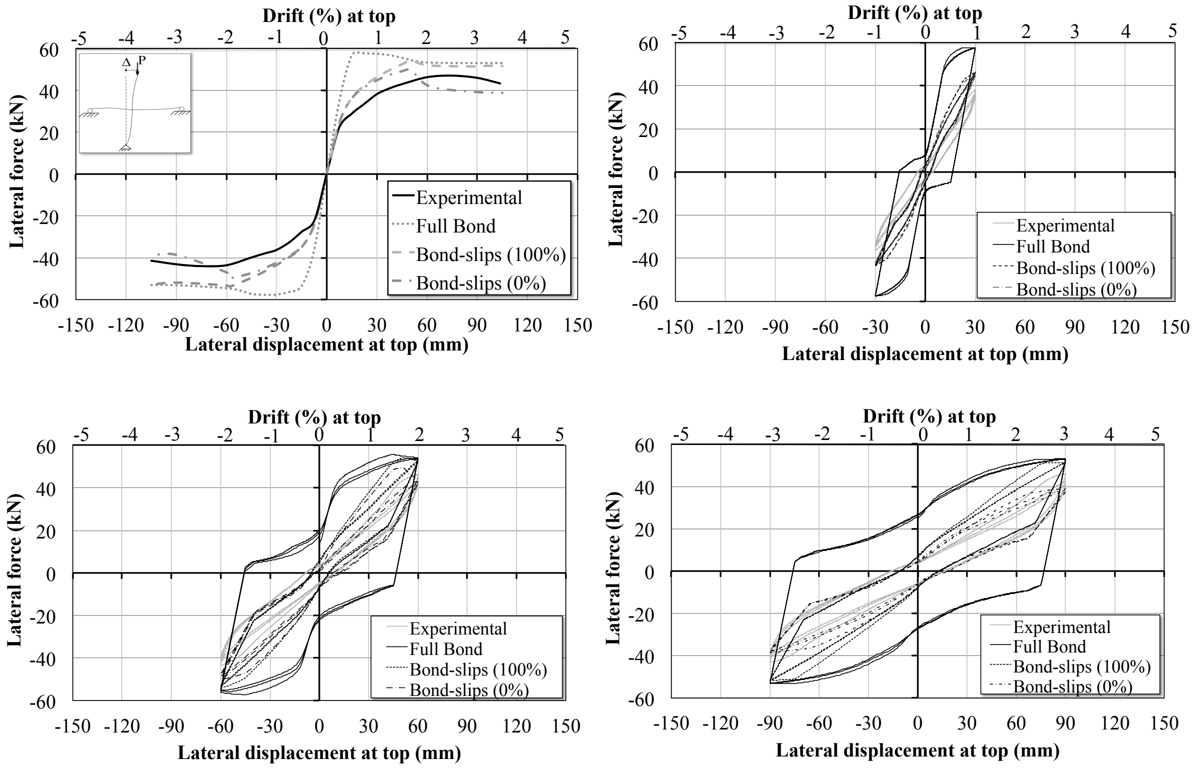

Figure 8: C11 Unwrapped (first test). Comparisons among the numerical and the experimental results.

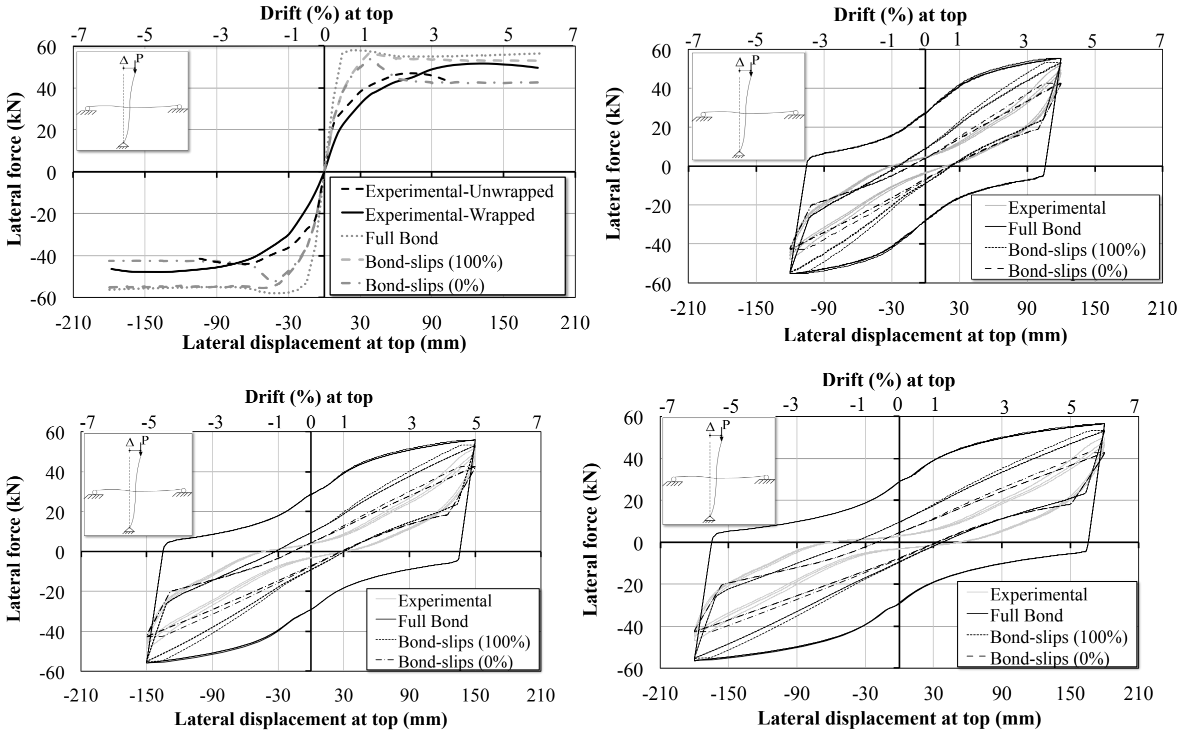

Figure 9: C11 Wrapped (second test). Comparisons among the numerical and the experimental results.

4.2 External joint

The numerical predictions obtained with the proposed model for the external joint are reported in Figure 10, where are as well compared with the experimental results.

In the numerical simulations an uniaxial stress-strain law incorporating bond-slips is assigned to the columns passing bars. The strength in tension and compression of diagonal trusses are calculated with the Mohr's circles approach. Two different sets of pinching factors are considered for the uniaxial hysteretic law: $p_x=1.0$, $p_y=1.0$ for simulating a full hysteretic behavior; and $p_x=0.5$, $p_y=0.0$ for accounting for a pinched hysteretic behavior, where p_x is the pinching factor for axial deformation, and p_y is the factor for the axial force, respectively. The pinching factors $p_x=0.5$, $p_y=0.0$ take into account the progressive regaining of compressive strength of diagonal cracks that, as experimentally observed, starts with a partial closure of diagonal panel cracks. Under this assumption, as it is easy to recognize, the numerical simulations are in good agreement with experimental results in terms of strength and of dissipated hysteretic energy (Figure 11).

Finally, in Figure 12 is reported a comparison of maximum and minimum diagonal cracks width numerically predicted with the ones measured during the experimental test of the external joint. The simulations provide a slight gap with respect to the experimental measurements, and an anticipation of the joint panel failure. This is very likely due to the column axial load eccentricity assumed into the numerical model, that reduces the specimen lateral strength and, therefore, anticipates the diagonal cracks initiation. Moreover, as it is clear to note the pinching factor do not influence the cracks width but only the dissipated hysteretic energy.

5 CONCLUSIONS

In this paper numerical simulations and comparisons with experimental results of existing RC beam-column sub-assemblages reinforced with smooth bars have been presented and discussed. The proposed models take into account the bond-slip phenomenon among the longitudinal bars and surrounding concrete, and the confinement of concrete when local strengthenings with FRP wraps are applied to columns ends. The proposed models are not time consuming and developed with simple criteria based on mechanical and geometrical elements properties, without any calibration procedure from an experimental data set.

The comparisons with the experimental results show a good agreement with the numerical predictions, and highlight that in old RC structures with smooth bars the lateral response, already from early cycles, is dominated by the reinforcements slippages and by the loss of anchorage of longitudinal bars passing through the joint panel.

The proposed models represent an useful tool for nonlinear analyses, since they require a limited numbers of additional elements for modeling the joint panel, and are capable of reproducing, with a good accuracy, the failure mechanisms experimentally observed of the internal and external beam-column joints.

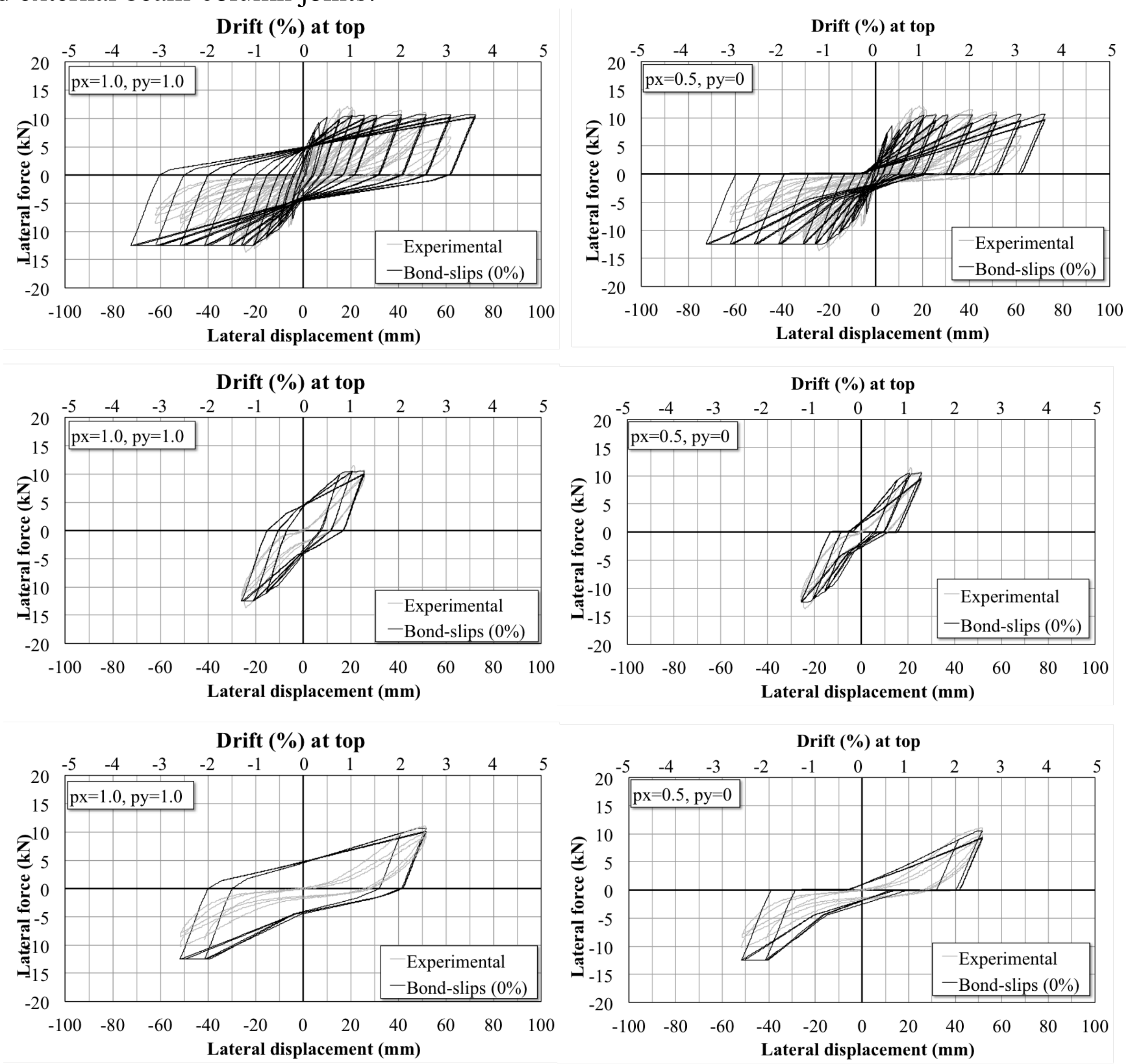

Figure 10: External joint T23. Comparisons among the numerical and the experimental results.

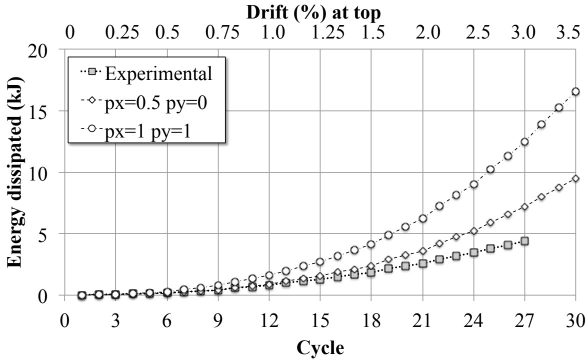

Figure 11: Comparisons among the dissipated hysteretic energy.

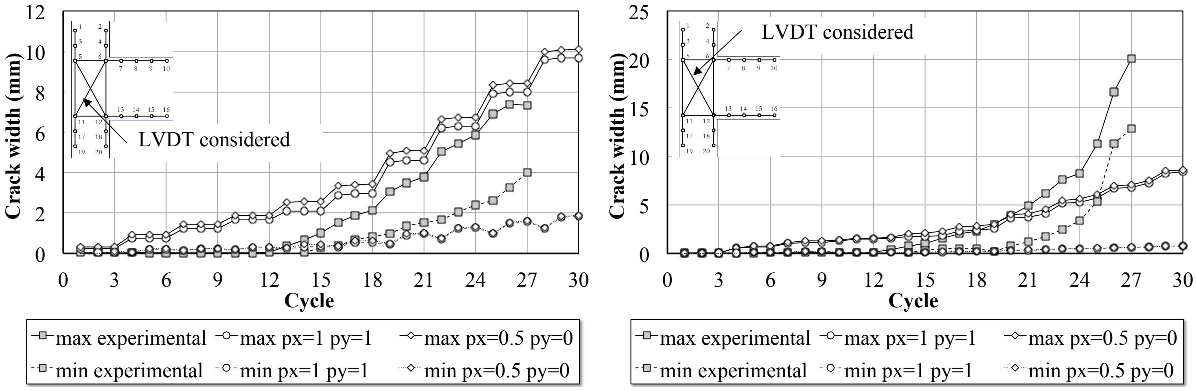

Figure 12: Comparisons numerical vs experimental panel cracks width.

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