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A NEW DISCRETE MACRO-NODE ELEMENT FOR THE SEISMIC BEHAVIOUR MODELLING OF REINFORCED CONCRETE FRAMES

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Abstract. *Recent earthquakes have demonstrated that the seismic behaviour of existing reinforced concrete buildings, not designed to resist to earthquake loadings, is strongly affected by the local collapses of the beam-to-column joints that can be subjected to shear collapse of the central region or bond-slip of the longitudinal steel bars. In this paper a new macro-node element able to account for the nonlinear behaviour of the beam-to-column joints is presented. The model can be represented by a simple mechanical scheme constituted by an articulated quadrilateral whose rigid edges connect beams and columns through nonlinear discrete interfaces and whose internal deformability is related to a single degree of freedom only. The model is able to account for the shear failure of the joint region as well as the flexural and shear plastic deformations at the contact edges of the connected elements allowing the nonlinear modeling of a frame structure through an assemblage of non-linear macro-nodes connected by means of elastic frame elements.*

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1 INTRODUCTION

Recent earthquakes have clearly shown the high vulnerability of existing reinforced concrete buildings not designed to resist to earthquake loadings. A reliable nonlinear model of a reinforced concrete building should include efficient simulation of beams, columns as well as beam-to-column joints. Many advanced numerical models have been proposed in the literature to simulate static, cyclic and dynamic behaviour of beams-columns elements also subjected to axial-moment-shear interactions [1,2,3] and some of these models have already implemented in advanced software largely used in engineering practice, however efficient models of beam-column joints still represent an academic issue whose results have not been made available to engineering community.

In this paper a new discrete macro-node element is proposed. The latter is able to simulate the nonlinear response of the node regions subjected to seismic loads with a limited computational effort, taking into account both the shear failure of the joint region, the flexural and shear plastic deformations at the edges of the elements and the bond-slip phenomena. According to the node capabilities, a nonlinear reinforced concrete frame structures can be modelled by means of the assembling of macro-nodes connected each others by elastic beam or column elements, provided that the plastic hinges can occur only at the ends of the frame elements. A brief description of the macro-node is presented and a first validation of full scale specimens, that have been the object of experimental campaigns [4,5] is reported. The results obtained so far seem to indicate that the proposed discrete macro-element node could be effectively used for the seismic assessment of reinforced concrete buildings and for the evaluation of possible retrofitting measures.

2 THE PROPOSED MACRO-MODEL

The proposed macro-node element has been inspired by a plane macro-element proposed by the authors for the simulation of nonlinear behaviour of unreinforced masonry buildings [6,7,8,9] and masonry infilled frames [10,11]. The basic element can be described by referring to a simple mechanical representation in which the element is regarded as an articulated quadrilateral endowed with along-side interfaces that, consistently to a fibre discretization, account for both the concrete and steel bars contributions. This macro-node possesses some similarities to the joint model proposed by Lowes and Altoontash [12], modified by Mitra, and Lowes [13] in order to simulate the behaviour of the joints without transversal steel reinforcements, afterwards calibrated and validated in [14-16]. However in the approach proposed herein the interfaces are discretized according to a detailed fibre discretization accounting for node and beams concrete and steel contributions.

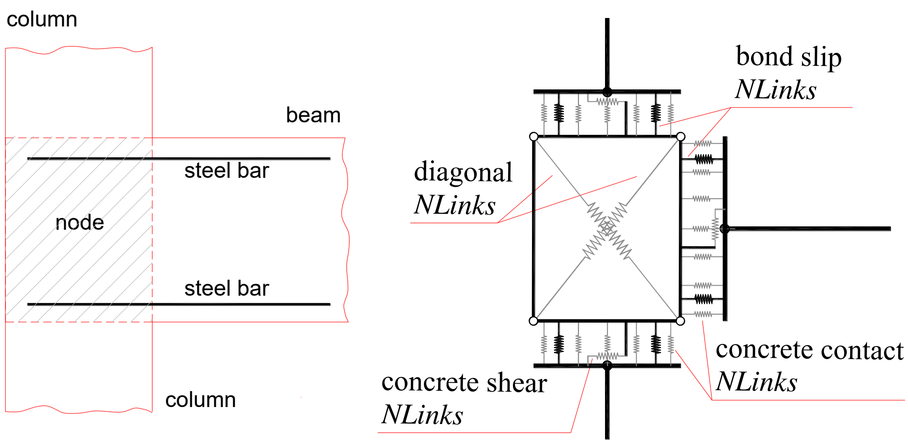


Figure 1. schematic representation of a beam-to-column joint and its macro-modeling.

The mechanical behaviour of the node is governed by the along-edge nonlinear interfaces and the in-plane deformability of the quadrilateral, related to a single degree of freedom, Figure 1. Aiming at adopting a uniform fibre discretization, the concrete behaviour is represented according to a regular distribution of nonlinear links (*concrete contact NLinks*) orthogonal to the interfaces, while the steel bar contribution is governed by concentrated nonlinear links (*bond slip NLinks*), Figure 1. The shear failure of the connected beams or columns is associated to the relative motion in the direction of the interfaces and described through a single longitudinal nonlinear link. The shear failure of the central core of the node is governed by the two diagonal nonlinear links (*diagonal NLinks*). The kinematics of the mechanical scheme, after a proper calibration procedure of the nonlinear links, allows an effective simulation of the typical failure mechanisms of the node as well as of the attached end beams and columns. It easy to recognise that degrees of freedom needed for the kinematics description of each macro-node are given by 4+3*n*, being *n* the number of the frame elements connected by the macro-node. A detailed description of the kinematic and the mechanical calibration are outside the aim of this paper whose goal is the presentation of the model. In the following section only the fundamental steps needed for a suitable calibration of the element are briefly summarised.

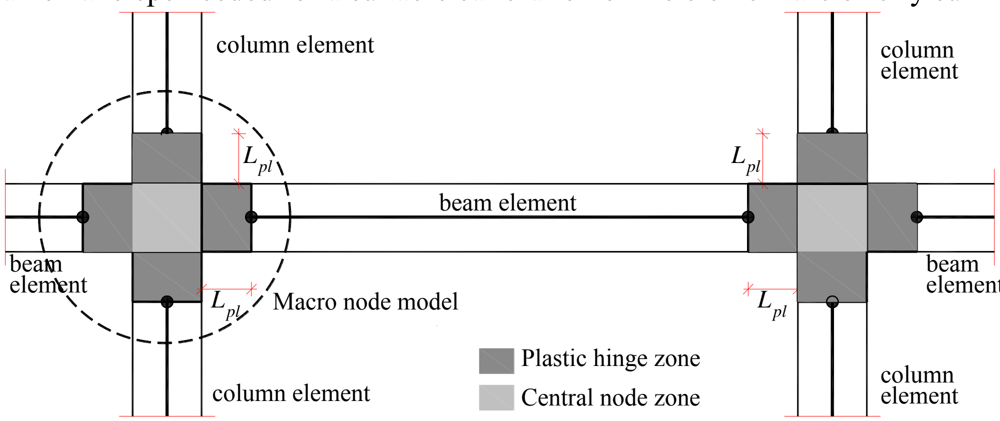


Figure 2. schematic assembling of a frame structure (a) and its representation by means of non-linear macro-nodes and linear elastic beams and columns elements (b).

3 THE MECHANICAL CHARACTERIZATION OF THE MACRO-NODE

In the following sub-sections the fundamentals of the mechanical characterization strategy of the model are briefly described.

3.1 The concrete model

In the applications reported in the following the Kent and Park uni-axial law is used to model the compressive behaviour of each fibre representing the concrete [17]. The corresponding constitutive law relationships can be expressed as two distinct phases: the pre-peak phase ($\varepsilon<\varepsilon_0$) and the post peak softening phase ($\varepsilon>\varepsilon_0$). The σ-ε constitutive law is reported in (1) where f_c is the compression strength, f_R the residual strength, ε_u the ultimate strain corresponding to f_R.

$$\begin{cases} \sigma = f_C\left[\dfrac{2\varepsilon}{\varepsilon_0}-\left(\dfrac{\varepsilon}{\varepsilon_0}\right)^2\right] & \varepsilon \le \varepsilon_0 \\ \sigma = f_C-\dfrac{f_c-f_R}{\varepsilon_u-\varepsilon_0}\left(\varepsilon-\varepsilon_0\right) & \varepsilon_0<\varepsilon\le\varepsilon_u \end{cases} \qquad (1)$$

The influence volume, associated to each link, is a function of the adopted fiber discretization, the joint dimension and the assumed length of the plastic hinge (L_{pl}).

3.2 The bond-slip behaviour of the steel bars

The presence of steel bars is modelled by means of nonlinear unidirectional links, collocated into the interface in the corresponding positions. The bond-slip occurrence is simulated by using a simplified model proposed by Braga et al. [18] (Figure 3a). The latter is characterised by a non-linear bond slip constitutive law characterized by three limit state points which correspond to the achievement of the tangential strength of the concrete (τ_{lim}) at the free edge (*point A*), the extension of the ultimate tangential stresses to the entire length of the bar (*point B*) and the achievement of the ultimate tangential stress at the hooked side of the bar (*point C*), respectively. After point *C* the system has constant residual stiffness coinciding with the hook stiffness k_s (Figure 3b).

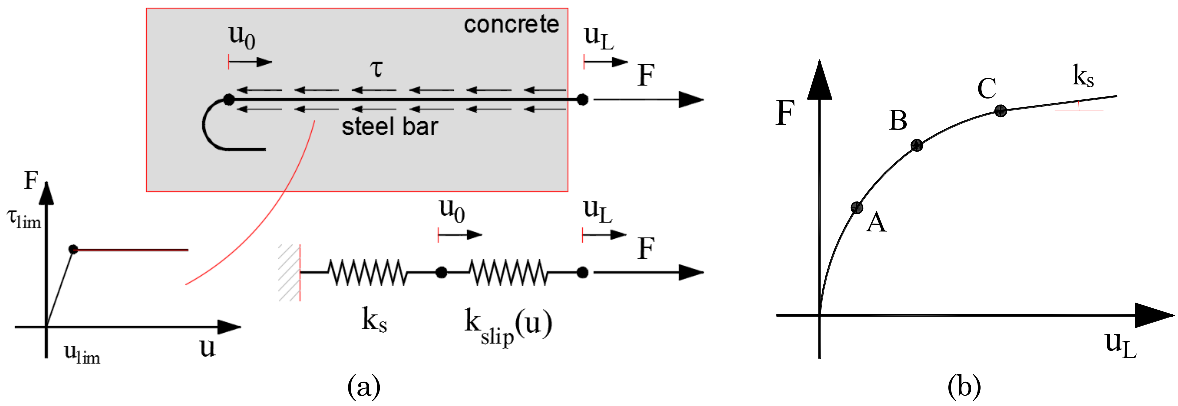


Figure 3. mechanical scheme of the bond-slip model from Braga et. al. [18] (a) and the corresponding bond-slip constitutive law (b).

In the macro-model each bar is modelled by two distinct links in order to simulate the anchorage in the node region (L_{node}) and the embedded length into the beam (L_{beam}). The two nonlinear links are connected at the interface level.

3.3 The shear behaviour of the joint

With reference to unreinforced joints, the shear load transfer through the central region of the node can be effectively modelled according to different models already proposed in the literature [19,20,21]. In the following the model proposed by Paulay et al. [22] and improved and experimentally validated by Mitra and Lowes [13] is considered. The joint shear behaviour is governed through an uni-dimensional constitutive law which rules the relationship between the generalised tangential stress (τ) and the generalised shear strain (γ). According to [23] the generalised shear stress is defined as $\tau=V/(b_j\ s)$, where V is the shear force acting at the horizontal edges of the joint, b_j is the width of the joint and s is the depth of the joint, defined as the minimum value among the depths of the columns and the beams (see Figure 4a). The dual generalised shear strain (γ) can be written as $\gamma=\delta_x/h_j+\delta_y/b_j$, where h_j is the eight of the joint; δ_x and δ_y respectively the horizontal and the vertical drift between the opposite edges of the joint (see Figure 4b).

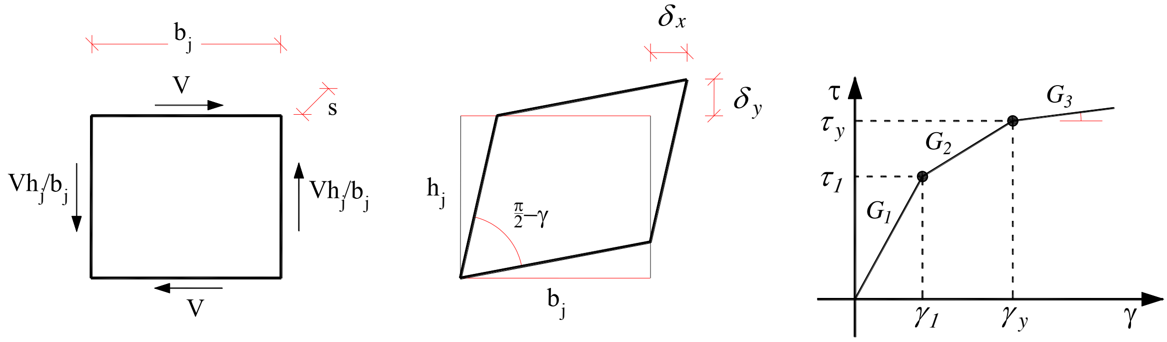


Figure 4. Mechanical behaviour of the internal joint: static characterization (a), deformed configuration (b), generalised constitutive law (c).

A tri-linear constitutive law, characterised by a crack point (τ_1, γ_1), a yielding point (τ_y, γ_y) and a constant post yielding module (G_3), as reported in Figure 4c, is adopted. This law is calibrated according to the procedure reported in [16] and here summarised in Table 1.

Table 1: Mechanical parameters of the joint shear constitutive law.

τ_1	τ_y	γ_1	γ_y	G_3
$0.48\sqrt{f_C}\,(Mpa)$	$2.08\sqrt{f_C}\,(Mpa)$	0.043%	0.6%	$0.015\dfrac{\tau_y}{\gamma_y}$

The diagonal links inherit the tri-linear constitutive law considered in Figure 4c, with crack point (u_1, F_1), yielding point (u_y, F_y) and residual stiffness (K_3). The latter parameters can be evaluated by imposing a simple equivalence in terms of global equilibrium and deformation. The results are summarised in Table 2, where $\alpha = \arctan(h_j/b_j)$ *and* $A = b_j s$ is the cross section area of the horizontal edge of the node.

Table 2: Mechanical calibration of the diagonal N-Links of the macro-node.

F_1	F_y	u_1	u_y	K_3
$F_1 = \dfrac{\tau_1 A}{2\cos(\alpha)}$	$F_y = \dfrac{\tau_y A}{2\cos(\alpha)}$	$\gamma_1 h_j \cos(\alpha)$	$\gamma_y h_j \cos(\alpha)$	$\dfrac{1}{2}\dfrac{G_3 A}{h_j}\dfrac{1}{\cos^2(\alpha)}$

4 VALIDATION OF THE MODEL

The applications reported in the following aims at providing a first numerical validation of the model by comparing some numerical results against already available experimental data. As a preliminary investigation, the analyses have been performed through a nonlinear-link-based FEM model implemented in the software SAP2000 [24]. Only static push-over analyses are carried out through a non-linear multi-step displacement controlled procedure.

4.1 Assembling of interior beam-to-column joint

The experimental research here considered has been performed by Walker and Alire [4,5] with the aim to investigate the behaviour of unreinforced concrete joints. Eleven sub assembling beam-to-column joint prototypes have been tested; all the specimens had the same geometry but different steel reinforcements in order to change the joint shear stress demand and to produce, alternatively, the beam flexural collapse or the joint shear failure. In the following only the specimen *SCDH-4150*, characterised by a shear joint collapse, is numerically investigated [5]. The test was carried out by imposing cyclic displacements at the free-end of each beams, directed in opposite directions(Figure 5).

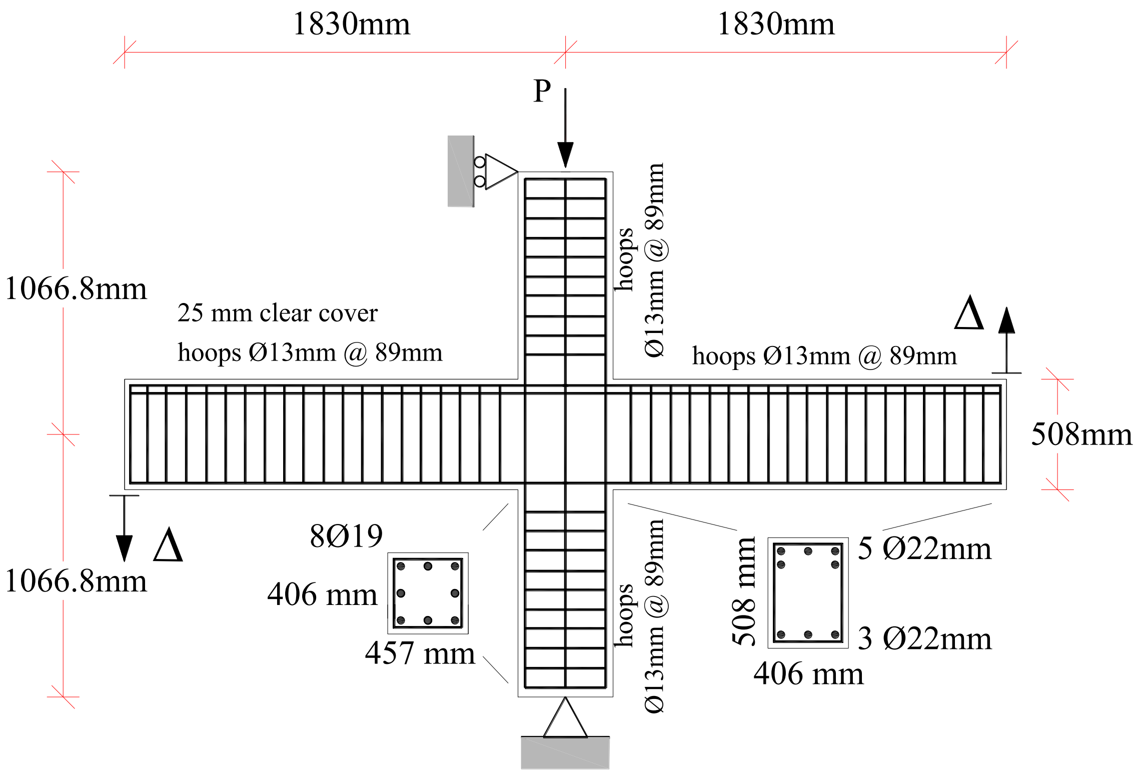


Figure 5. *SCDH-4150* test: setting, geometry of the specimen and steel bars reinforcement details.

A constant axial load (*P=641 kN*) equal to the 10% of the ultimate compression load of the column, was applied at the top section of the columns. The test-setting, the geometry of the prototype and the steel reinforcement bars are reported in Figure 5, while the mechanical parameters adopted for the concrete and steel are reported in Table 3.

Table 3: Mechanical parameters of the materials.

Concrete				Steel bars		
f_c [Mpa]	f_R [Mpa]	ε_0 [-]	ε_u [-]	f_y [Mpa]	E [Mpa]	ε_{su} [-]
34.5	6.8	0.0022	0.02	462	200000	0.1

The numerical results obtained by the macro-model are reported in Figure 6 in terms of the stress vs strain of the central panel of the macro-node. The results are compared to the cyclic response obtained experimentally. It can be observed that the response of the macro node well approximates the experimental envelope curve.

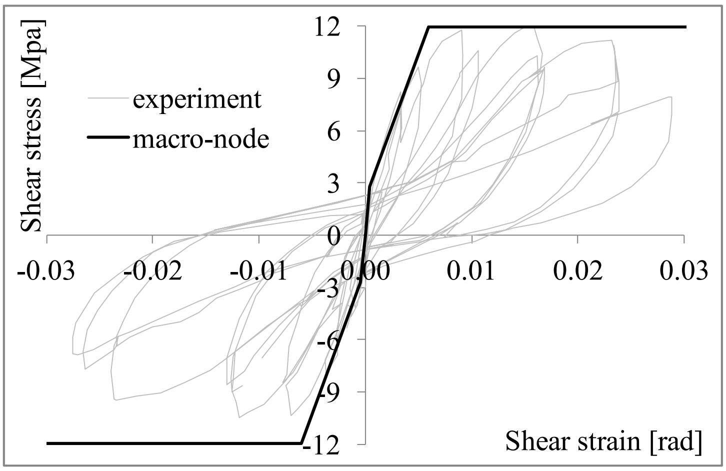


Figure 6. *SCDH-4150* test: numerical capacity curve and experimental results.

A further comparison is performed in terms of collapse mechanism and reported in Figure 7. In particular, Figure 7a shows the experimental damage on the joint showing a complete failure of the joint region; while in Figure 7b the deformed mesh of the macro node model is shown. It can be observed that the joint deformation contributes significantly to the total deformation of the structure.

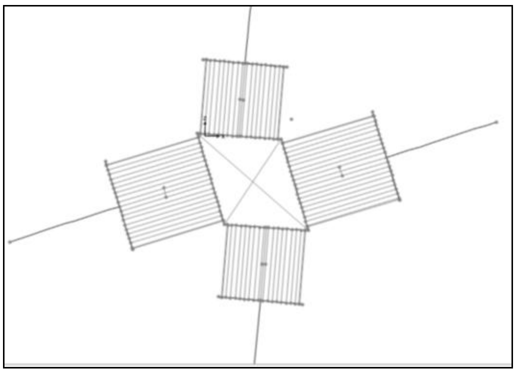
(a)

(b)

Figure 7. Experimental (a) and numerical (b) failure mechanism.

5 CONCLUSIONS

Recent seismic events have demonstrated that the seismic behaviour of existing reinforced concrete buildings is strongly affected by the local collapses of the beam-to-column joints that can be subjected to shear collapse of the central region or bond-slip of the longitudinal steel bars. Even though many advanced numerical models are available in the literature an explicit modelling of the joints in the every-day engineering practice, mainly in large structures, still remains an open issue.

In this paper a macro-node element able to account for the nonlinear behaviour of the beam-to-column joints is presented. The model can be represented by a very simple mechanical scheme constituted by an articulated quadrilateral whose rigid edges connect beams and columns through nonlinear fibre interfaces. The model is able to account for the shear failure of the joint region, the flexural and shear plastic deformations at the contact edges of the joint-elements and the bond-slip of the steel bars. A great advantage of the proposed model is that the nonlinear behaviour of a frame structure can be obtained as an assemblage of non-linear macro-nodes connected by means of elastic beam or column FEM elements. The numerical simulations here reported and the versatility of the element seem to confirm the potentiality of the proposed model to be adopted for an efficient simulation of reinforced concrete frame structures for which the deformability or the failure of the joint regions play a key role in the overall nonlinear behaviour of structure.

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