A DISCRETE MACRO-NODE FOR MODELING THE SEISMIC BEHAVIOUR OF R/C BEAM TO COLUMN JOINTS

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ABSTRACT

The seismic ductility performance of an existing reinforced concrete building, not designed to resist earthquake loadings, can be strongly affected by local collapses of the beam-to-column joints. A premature failure of a joint region can represent a further limitation of the connected beams and columns ductility resources. Consequently, the accurate modelling of the non-linear behaviour of joint regions represents a fundamental aspect for obtaining a reliable seismic assessment of an existing reinforced concrete frame structure. In this paper a new macro-node element able to account for the nonlinear behaviour of beam-to-column joint is presented. The joint element can be represented according to a simple mechanical scheme corresponding to an articulated quadrilateral, whose rigid edges connect beams and columns through nonlinear discrete interfaces, and whose internal shear deformability is related to a single degree of freedom only. The proposed beam-to-column joint model is able to account for the shear failure of the joint region as well as the flexural and shear inelastic deformations of the connected beams and columns ends. This novel capability of the joint element allows the non-linear modeling of a frame structure as an assemblage of non-linear macro-nodes connecting elastic frame elements.

Keywords: reinforced-concrete frame structures, beam-to-column joints, macro-modeling approach, seismic vulnerability assessment, fiber section approach.

1. INTRODUCTION

Many reinforced concrete buildings designed for vertical and wind loads have been realized in regions that only recently have been recognised as seismic prone. Many earthquakes have recently underlined the high seismic vulnerability of these modern existing buildings whose seismic assessment constitutes a priority for the evaluation of the seismic risk in many earthquake prone region on the world. A reliable seismic assessment of an existing structure is however related to an accurate nonlinear model of construction that should include all the elements that can influence the nonlinear response of the building when subjected to earthquake loadings. For a reinforced concrete existing building a reliable model should include efficient simulation of beams, columns as well as beam-to-column joints and non-structural infills (Panto et al. 2014, Panto et al. 2017). Many advanced numerical models have been proposed in the literature to simulate the nonlinear static, cyclic and dynamic behaviour of beams and columns (Izzudin et al. 1994, Spacone et al. 1999, Pantò et al. 2017a, Pantò et al. 2017b). Several advanced nonlinear models have already implemented in engineering software most of them largely used in structural design practice. However the current research on the modelling of beam-column joints still represent a research academic issue and the results obtained so far 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

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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 as well as the bond-slip phenomena. According to the new node capabilities, a nonlinear reinforced concrete frame structures can be modelled by means of the assembling of macro-nodes simply connected each other’s by elastic beam or column frame elements, provided that the plastic hinges can occur only at the ends of the frame elements. The macro-node element can be represented according to a simple mechanical scheme corresponding to an articulated quadrilateral, whose rigid edges connect beams and columns through nonlinear discrete interfaces, and whose internal shear deformability is related to a single degree of freedom only. In this paper after a brief description of the proposed macro-node, a first validation of the capability of the element to be adopted for nonlinear static analysis is performed by simulating the results of experimental campaigns conducted by Walker (2001) and Alire (2002) on plane reinforced concrete frames. 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 MACRO-NODE FOR MODELING THE BEAM-TO-COLUMN REGION

The proposed macro-node for modelling the beam-to-column region has been inspired by a plane macro-element proposed by the authors for the simulation of nonlinear behaviour of unreinforced masonry buildings (Caliò et al. 2005), (Caliò et al. 2012), (Pantò et al. 2016), (Pantò et al. 2017c) and masonry infilled frames (Caliò et al. 2014), (Pantò et al. 2017a), (Pantò et al. 2017b). 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 alongside 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 (2003), modified by Mitra and Lowes (2004) for the simulation of the nonlinear behaviour of the joints without transversal steel reinforcements. The same joint element has been subsequently calibrated and validated in by Lehman (2004), Mitra and Lowes (2007) and Anderson et al. (2008). However in the modelling approach proposed herein the joint interfaces are discretized according to a detailed fibre discretization that accounts for the behaviour of the central node region as well as the inelastic response of the beams and columns ends connected to the joint.

The mechanical behaviour of the node, Figure 1, is governed by the along-edge nonlinear interfaces and the in-plane deformability of the quadrilateral, related to a single degree of freedom. Aiming at adopting a uniform fibre discretization, the concrete behaviour is represented according to a regular distribution of nonlinear links (concrete $N_{Links}$) orthogonal to the interfaces, while the steel bar contribution is governed by concentrated nonlinear links (steel bar $N_{Links}$) able to simulate also the activation of bond slip, 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 two diagonal nonlinear links (diagonal $N_{Links}$). 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 ends of beams and columns. The degrees of freedom needed for the kinematics description of each macro-node are given by $4+3n$, 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 conference paper whose goal is the introduction of the model and a first validation of its capabilities. In the following section only the fundamental steps needed for a suitable calibration of the element are briefly summarised.
The model is able to account for the failure mechanisms of the joint as well as the flexural and/or shear plastic damage at the end sections of the beams or columns connected. By using the proposed numerical strategy, the nonlinear behaviour of a frame structure can be simulated through an assemblage of nonlinear macro-nodes simply connected by means of elastic frame elements. In Figure 2 a qualitative representation of a simple 1-bay 1-storey frame is shown. Figure 2a reports the geometrical layout of the discretization of the joint regions and the beam and columns ends. Figure 2b illustrates the corresponding macro-nodes and elastic frame elements discretization, the interface lengths $L_p$ are representative of the length of the plastic hinges at the ends of beams and columns.

3. 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 Interface concrete N-Links

In the applications reported in the following the Kent and Park uni-axial law is used for modelling the compressive behaviour of each fibre representing the concrete (Kent and Park 1971). The corresponding constitutive law relationships can be expressed as three distinct phases: a non-linear parabolic phase ($\varepsilon < \varepsilon_0$) up to the ultimate compression strength $f_c$; a post-peak linear softening phase ($\varepsilon_0 < \varepsilon < \varepsilon_u$) and a perfect plastic phase corresponding to a residual strength $f_R$.

The influence volume, associated to each link, is a function of the adopted fiber discretization of the beam ($\lambda$), of the joint dimensions ($b_j, h_j$) and the assumed length of the plastic hinge ($L_p$) (Figure 3a). The stiffness of the Links is calibrated considering the length $L_p$ plus half length of the central node (Figure 3b).
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. (Braga et al. 2012) in which a constant tangential strength (τ_{lim}) is considered for the concrete (Figure 4a).

The global bond-slip constitutive behaviour is characterised by a tri-linear constitutive law (Figure 4b). The yielding point (F_y, u_y) corresponds to the achievement of the limit tangential strength of the concrete (τ_{lim}) at the loaded end of the bar, while the ultimate strength point (F_u, u_u) corresponds to the achievement of the ultimate tangential strength at the free side of the bar. It is assumed that after this point the system has zero residual stiffness until the reaching of a displacement identifying the failure followed by a softening ending with a residual low resistance, Figure 4b. 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. The yielding and ultimate bond-slip force of the steel bar are reported in the expressions (1) and (2) respectively, where, D is the diameter of the bar, L the anchorage length and L_0 the critical length of anchorage.

\[
F_y = \frac{1}{2} \tau_{lim} \pi D L_0 \quad \text{if } L \geq L_0
\]

\[
F_y = \frac{\tau_{lim} \pi D L}{2} \left[ \frac{3E_y D u_{lim} - 2L^2 \tau_{lim}}{3E_y D u_{lim} + 4L^2 \tau_{lim} + 1} \right] \quad \text{if } L < L_0
\]

\[
F_u = \tau_{lim} \pi D L
\]
3.3 The shear behaviour of the joint

With reference to unreinforced joints, typical of buildings not designed to resist to earthquake loadings, the shear load transfer through the central region of the node can be effectively modelled according to different methods already proposed in the literature (Alath and Kunnat 1995), (Biddah and Ghobarah 1999), (Shin and Lafave 2004). In the applications reported in the following the model proposed by (Paulay et al. 1978) modified and experimentally validated by (Mitra and Lowes 2004) is considered. The joint shear behaviour is governed through a uni-dimensional constitutive law ruling the relationship between the generalised tangential stress, $\tau$, and the generalised shear strain, $\gamma$.

According to Park and Mosalam (2012) the generalised shear stress is defined as $\tau = V / (s \cdot b_j)$, being $V$ the shear force acting at the horizontal edges of the joint, $b_j$ the width of the joint and $s$ the depth of the joint, defined as the minimum value among the depths of the columns and the beams. The corresponding generalised shear strain $\gamma$ can be written as $\gamma = \delta / h_j + \delta / b_j$, where $h_j$ is the eight of the joint; $\delta$ and $\delta_j$ respectively the horizontal and the vertical offset between the opposite edges of the joint. A tri-linear constitutive law, characterised by a crack point $(\tau_1, \gamma_1)$, a yielding point $(\tau_y, \gamma_y)$ and a constant post yielding module $(G_3)$, Figure 5, is adopted. This law is calibrated according to the procedure reported in (Anderson 2008) and here summarised in Table 1.

![Figure 5. One-dimensional constitutive law associated to the diagonal links.](image)

The diagonal links inherit the tri-linear constitutive law considered in Figure 5, 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 equations (3) with reference to the crack and yielding forces, in equations (3) with reference to the corresponding displacements and in equation (4) with reference to the residual stiffness.

\[
F_i = \frac{\tau_i A}{2 \cos(\alpha)}; \quad F_y = \frac{\tau_y A}{2 \cos(\alpha)} \tag{3}
\]

\[
u_j = \gamma_j h_j \cos(\alpha) ; \quad u_i = \gamma_i h_j \cos(\alpha) \tag{4}
\]

\[
K_j = \frac{G_3 A}{2 h_j \cos^2(\alpha)} \tag{5}
\]

<table>
<thead>
<tr>
<th>$\tau_1$ (Mpa)</th>
<th>$\tau_y$ (Mpa)</th>
<th>$\gamma_1$ (%)</th>
<th>$\gamma_y$ (%)</th>
<th>$G_3$ (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.48 $\sqrt{f_c}$</td>
<td>2.08 $\sqrt{f_c}$</td>
<td>0.043</td>
<td>0.6</td>
<td>0.015 $\frac{\tau_y}{\gamma_y}$</td>
</tr>
</tbody>
</table>

Table 1. Mechanical parameters of the joint shear constitutive law.
4. NUMERICAL APPLICATIONS

In a previous paper (Panto et al 2017) the same authors provided a simple validation of the model by comparing some numerical results on a single beam-to-column joint against already available experimental data. In this paper the proposed macro-node is applied to simulate the nonlinear behaviour of a plane three-storey three-bay frame characterised by constant cross sections 300x457mm and 300x500mm respectively for columns and beams and the same reinforcing steel bars considered in (Pantò et al. 2017). A simple geometry is considered (Figure 6a) with 5.00m span length, 0.3m constant transversal width, 3.5m first-storey height and the others 3.2m height. A uniform vertical load of 10.0 kN/m is applied on the beams (Figure 6b) at all levels.

A first comparison between a classical frame elements discretization (in which the beam-to-column joints are simply modelled as rigid links considering offsets of the central node) and a model obtained as an assemblage and macro-node and beams, both implemented in SAP, are performed in the elastic field. Figure 7 reports the first three periods and modes of vibration of the two modelling strategies showing satisfactory coherence between the two models in the elastic field.

Figure 6. Geometrical scheme of the frame (a) and 2D view of the SAP model with the applied vertical loads (b).

Figure 7. First three mode shapes and periods of vibration corresponding to the standard frame modelling approach (a) and to the macro-node assembly (b).
In order to investigate the influence of the different damage mechanisms of the macro-node frame assembly on the global nonlinear behaviour and failure mode of the structure, a push-over analysis is performed considering a lateral loads distribution proportionally to the masses. Three different models (A, B and C) are considered, by changing two mechanical parameters: the concrete compression strength and the anchorage length of the steel bars. **Model A** is characterised the mechanical properties reported in Table 2, and an anchorage length of the steel bars sufficient to avoid the bond slip. **Model B** differs from model A because is characterised by a very low value of concrete strength, $f_{c}=12,0$ Mpa. **Model C** is characterised by the same low value of concrete strength of model B, $f_{c}=12,0$Mpa, and an anchorage lengths of steel bar 0,5m, that is insufficient to avoid bond slip behaviour. The results of the push-over analyses performed on these three models are reported in Figure 8 where the top displacement of the structure is expressed as a function of the global base shear. The corresponding collapse mechanisms of the investigated frames are shown in Figure 9.

![Figure 8. Capacity curves of the three macro-node models.](image)

The comparison of the results clearly show the capability of the model to simulate the reduction of capacity of structure related to the occurrence of damage in the joint regions as well as in the beams and columns ends. The failure mechanisms, reported in Figure 9 also highlights the different distribution of damage related to the three considered assumptions. In presence of a low value of concrete strength and good anchorage length of the steel bars (Model B), the two central joints at the first level exhibited a shear collapse. Decreasing the anchorage lengths of the steel bars (Model C) or considering a high value of concrete strength (Model A), the flexural collapse of the beams anticipated the joint shear collapse. Beside the deformed shapes, a representation of the link forces at the last step of the analysis is reported for each investigated model. It is possible to observe that in models A and B the joint-diagonal links are subjected to higher forces if compared to model C, where the premature flexural collapses of the beams, due to the bond slip mechanisms, characterised the ultimate state of the structure.

![Ultimate deformed shape](image)

![Link axial forces(blue=traction; red compression)](image)

Model A
The results obtained so far confirm the high potentiality of the proposed model to be adopted for investigating existing reinforced concrete frame structures for which the deformability and strength of the joint regions as well the bond-slip failure of the steel bars play a key role in the overall nonlinear structural behaviour.

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. Although many advanced numerical models are available in the literature, an explicit modelling of the joints in the every-day engineering practice 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 with rigid edges which 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 nonlinear macro-nodes connected by means of elastic beam or column FEM elements. The numerical simulations 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.


