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Nonlinear modeling of a RC beam-column connection subjected to cyclic loading

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Abstract. When reinforced concrete structures are subjected to strong seismic forces, their beam-column connections are very susceptible to be damaged during the earthquake event. Consequently, structural designers try to fit an important quantity of steel reinforcement inside the connection, complicating its construction without a clear justification for this. The aim of this work is to evaluate –and demonstrate- numerically how the quantity and the array of the internal steel reinforcement influences on the nonlinear response of the RC beam-column connection. For this, two specimens (extracted from an experimental test of 12 RC beam-column connections reported in literature) were modeled in the Finite Element code FEAP considering different stirrup's arrays. The nonlinear response of the RC beam-column connection is evaluated taking into account the nonlinear thermodynamic behavior of each component: a damage model is used for concrete; a classical plasticity model is adopted for steel reinforcement; the steel-concrete bonding is considered perfect without degradation. At the end, the experimental responses obtained in the tests are compared to the numerical results, as well as the distribution of shear stresses and damage inside the concrete core of the beam-column connection, which are analyzed for a low and high state of confinement.

Keywords: beam-column connection; reinforced concrete; nonlinear material behavior; finite element

1. Introduction

In civil engineering construction, Reinforced Concrete (RC) is one of the most important hybrid materials used widely and its efficiency depends on different aspects related to structural design as well as to constructive techniques. In both cases, the structural security of RC structures is guaranteed by the accomplishment of the local regulatory requirements (in other words, design rules and construction codes, as it is reviewed by Sasmal and Ramanjaneyulu 2012): unfortunately, due to the complexity of RC, these requirements adopt a lot of technical simplifications to reduce the effects of uncertainties. In the case of structural design, the accomplishment of the required level of security is done by specifying the geometrical dimensions of the structural element (the concrete body) as well as the quantification and location of the respective internal steel reinforcement (as an example see confinement recommendations of ACI Committee 318 2005).

The observation of these specifications must assure that the structural element will develop the expected loading capacity, based on a good transference of internal efforts and stresses between concrete and steel bars (a good description of the distribution of internal forces inside the joint is done by Zhou and Zhang 2012). Because of this, some researchers have focused in improving the structural response of the joint using anchor-type intermediate bars and advanced details of doubly confined closed stirrups in the beam near the joint (Ha and Cho 2008). Despite ongoing research, it is very common that a blind application of these design specifications complicates unnecessarily the construction layout of the structural elements, especially the layout of the beam-column connections, which are at the same time, the key-points for the structural stability of the whole system. To get around this, some building contractors try to reduce the quantity of reinforcement leaned on a reinterpretation of the standard codes: for example, taking account of the ACI's recommendation (ACI Committee 318 2005) that stipulates that joint strength is a function of only the compressive strength of the concrete and requires just a minimum amount of transverse reinforcement in the joint -which is true solely if the effective shear area is less than the column cross-sectional area-, it is possible to rearrange the layout of the internal reinforcement of the beam-column connection. Nevertheless, removing any steel rebar in an unreasoned way might reduce dramatically the resistance of the joint, particularly in the event of an earthquake, and this situation might drive to search new –and expensive- ways of rehabilitation/reparation (Wang 2012, Karayannis and Sirkelis 2008, Ha *et al.* 2012).

Being the beam-column connection the main point of transmission of forces between horizontal elements (beams) and vertical elements (columns), it should provide enough stiffness to the global structural system and consequently, there is a high concentration of stresses inside the connection that potentially might produce any damage in concrete and/or plastic deformations on steel bars. Some

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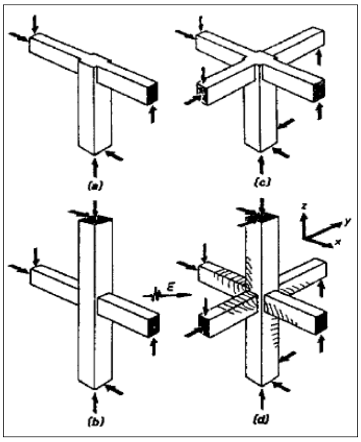
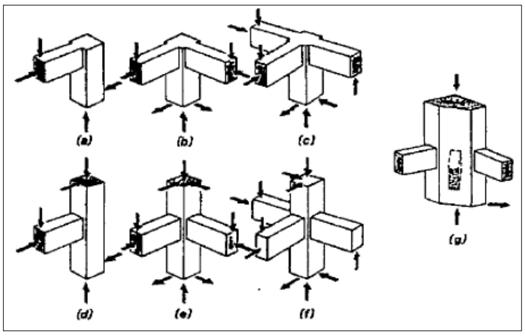


(a) with internal joint



(b) with external joint

Fig. 1 Classification of beam-column connections according to Alcocer (1991)

recent experimental works have focused in determine which parameters can affect the shear resistance of this kind of joints (Kim and LaFave 2007, Wong and Kuang 2011). That is the reason why beam-column connection is one of the riskiest points of failure in RC structures.

In the other side of the story, the modern design is becoming strongly dependent of the numerical method adopted for structural analysis –typically a standard finite element code- and the prediction of the realistic response (efforts and displacements) is directly derived from the computational capabilities of the selected software. A non-initiated engineer in numerical simulations may believe that modeling of any mechanical problem concerns only the definition of a set of load combinations, forgetting that a numerical model should include as well: the choice of a proper finite element, the association to an efficient material model, and a good representation of the real boundary conditions. Paradoxically, even if the computational resources become powerful and user-friendly, the local study of any RC connection is practically disregarded by structural engineers -maybe for the complexity of preparing a full detailed modeling-, while steel reinforcement array is basically proposed from practical recommendations extracted from limited experimental tests. In consequence, the quantity of steel reinforcement inside the connection might be overestimated or simply poor distributed. In general, the Beam-Column (B-C) connections can be classified following two criteria (see Alcocer 1991):

- By the geometrical configuration of the steel reinforcement,
- By the local behavior of the full connection.

According to the first criterion, there are B-C connections with internal joints -when the beam's steel bars pass across the joint (see Fig. 1(a)) - and B-C connections with external joints -when the beam's steel bars are anchored inside the joint (see Fig. 1(b)). Based on the second criterion, there are elastic B-C connections (that means, any plastic behavior occurs out of the joint) and inelastic B-C connections (if any nonlinear phenomenon appears into the joint).

In conventional structures, the beam-column connection must be designed not only against the development of any nonlinear phenomenon inside, but also it should induce the failure out of the connection: the most accepted criteria of failure for the connected members is the SC-WB (Strong Column – Weak Beam), which means that if any plastic articulation is developed in the structural system, it should appear on the beam instead of on the column (Visintin *et al.* 2012).

In spite of these recommendations, the B-C connection might fail, and the most common mechanisms of failure identified by different authors (Ma *et al.* 1976, Meinheit and Jirsa 1977, Lowes and Moehle 1995, Lowes 1999, Lowes *et al.* 2004) are the following:

- Beam reinforcement anchorage is not enough inside the joint and the bar slips,
- Shear forces developed into the joint activate the inelastic response of the core of concrete.
- A poor transference of shear forces may produce a failure plan between the joint and the beam, or between the joint and the column.

The aim of this work is to evaluate numerically how the quantity and the array of the steel reinforcement inside the RC beam-column connection might affect its structural response when this one is subjected to cyclic loading: For this, different stirrup's arrays and quantities will be modeled and analyzed. In these simulations, the nonlinear response of the RC beam-column connection was evaluated taking into account the nonlinear thermodynamic behavior of each component: for concrete, it was adopted a concrete damage model proposed by Mazars (1986), which is able to reproduce the non-symmetric behavior of concrete (concrete compression strength differs to traction strength), as well as the particular dissipative effects due to the crack closure during the cyclic loading. For steel reinforcement, we used a classical elasto-plastic model based on the Von Mises criterion, in order to reproduce the inelastic unidimensional deformation of the steel bars induced by traction. In all of the cases, the steel-concrete bonding was considered perfect –that means, without any degradation-, based on two assumptions: a) even if non-perfect bonding becomes interesting to evaluate a realistic inelastic response of reinforced concrete as soon as cracking evolves (Dominguez *et al.* 2005, Ibrahimbegovic *et al.* 2010), for a first non-detailed approach we consider redundant its influence on joint's elastic shear resistance; b) the slip and decohesion between stirrups and concrete are not activated on the early stages of cracking.

In order to build a realistic model, as a reference it was adopted the experimental results reported by Alamedinne *et* *al.* (1991) for a RC beam-column connection, although other good candidates to be modeled in the future are: the experimental campaign of (Kai and Li 2012), who focused in studying the dynamical performance of RC beam-column substructures with an initial damage; and the experiments on RC beam-column joints performed by Sharma *et al.* (2010) who characterized the relationship between ductility and failure modes during a cyclic loading. The numerical simulations were made in the Finite Element code FEAP (Taylor 2005), in which the concrete damage model was implemented through the use of a user material subroutine.

2. Basis of the nonlinear modeling

2.1 The experimental test of reference

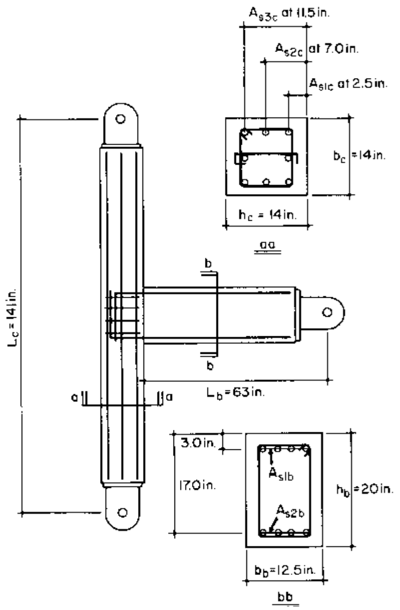
For the numerical study, the experimental work carried out by Alameddine and Ehsani (1991), was taken as a reference. It consisted in obtaining the structural response of an external beam-column joint subjected to cyclic loading, in order to verify the recommendations of the ACI-ASCE-352 code. The researchers classified the tests in three sets of four specimens, each set with a specific concrete high resistance. In all of the tests, three variables were observed and studied:

a) The compression strength of concrete (55.8 MPa (8 ksi), 73.8 MPa (11 ksi) and 93.8 MPa (14ksi) respectively);

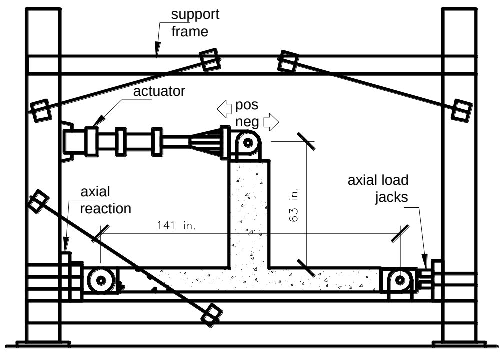
b) The maximal value of the shear stress into the connection, with a minimal value of 7.6 MPa (1100 psi) and a maximum of 9.7 MPa (1400 psi); and

c) The contribution of the stirrups by improving the confinement of the core of concrete (see Table 1 for stirrup characteristics).

Each specimen was designated by two letters and a number, indicating the level of the maximal joint shear stress (first letter), the level of confinement induced by the number of stirrups (second letter) and the value of the compressive strength. For example, the LH11 denomination designates a specimen with a "*Low*" shear stress (L), "*High*" confinement level (H), and a compressive strength of 11 ksi (number 11).



(a) dimensions of the specimen, in inches



(b) mounting of the experimental test

Fig. 2 Description of the experimental test of Alameddine and Ehsani (1991)

Table 1 Reinforcement of the transversal section of specimen's elements

Specimen	LL	LH	HL	HH
A_{s1c}	2#8, 1#7	2#8, 1#7	3#8	3#8
A_{s2c}	2#7	2#7	2#8	2#8
A_{s3c}	2#8, 1#7	2#8, 1#7	3#8	3#8
A_{s1b}	4#8	4#8	4#9	4#9
A_{s2b}	4#8	4#8	4#9	4#9
Number of stirrups	4	6	4	6
r_t	1.2	1.8	1.2	1.8
$h_s/d_{b,col}$	20	20	20	20
Development length l_{dh} (inches) required for $f'c$=8,000 (psi) (Recommendations 1985)	8.9	8.9	10.0	10.0
Development length l_{dh} (inches)	10.5	10.5	10.5	10.5

Notes: 1 psi=6.89 kPa; 1 inch=25.4 mm; L: Low; H: High; the first letter indicates the level of shear stress; the second letter indicates the level of confinement.

The geometrical characteristics of the specimen are shown in Fig. 2(a). Applying a cyclic controlled displacement test (see Fig. 2(b)), the initial displacement in the free edge of the beam was of ±½ inches (13 mm), being increased in ±½ inches (13 mm) in each cycle of loading; during the test a small axial load was applied in the top of the column. At the end of the test, they reached distortions up to 7% corresponding to a maximum displacement of 4½ inches (114 mm), concluding that:

- Elevated shear stresses reduce significantly the load capacity of the connection.
- The value of the ultimate shear stress recommended by the ACI-ASCE-352 for the joint was lower than the value observed in the experimental tests for high resistance concrete.
- The increment of transversal reinforcement reduces the deterioration into the connection, avoiding the failure of the reinforcement anchorage.

Concerning to the expected ultimate loading capacity of each RC beam-column connection, it was considered that this condition occurs when tension stress in the steel reinforcement reaches a value 25% higher than the nominal yield stress of the reinforcing steel. Specimens with a low shear level and high joint confinement were able to develop the ultimate capacities in the beams with a variation of ±3.8% from the predicted capacities. In addition, these same specimens had the least stiffness degradation and loss of load-carrying capacity at displacement beyond the yield displacement.

2.2 Description of the thermodynamic nonlinear models adopted for numerical simulations

Since the point of view of the experimental researchers (Alamedine and Ehsani 1991), their results show that concrete compressive strength and the degree of confinement are both the most important parameters that controls the shear capacity of the beam-column connection and its ductility, and secondly the steel reinforcement quantity. In our case, the main objective is to evaluate how the shear steel reinforcement influences the structural response of a B-C connection, but it is possible to corroborate numerically if compressive strength and confinement are the most sensitive parameters, as it is done in Ibrahimbegovic and Brancherie (2003). For this, it is necessary that numerical simulations rely not only on searching a set of robust nonlinear models that reproduce the realistic behavior of each concerned material, but also on identifying the corresponding material model parameters, what could be a little complicated when the experimental tests are done separately from the numerical models. In those cases, the numerical model's precision and accuracy depends directly on a limited set of experimental data. As a first approach, in this work we concentrated exclusively in defining a set of nonlinear models based on the elastic parameters provided by the experimental researchers: in order to accomplish with this challenge, some respected nonlinear models based on a thermodynamic formulation were adopted for each material (a damage model for concrete; a classical plasticity model for steel) and combined into the model, in order to include the effects of the different dissipative phenomena associated to each inelastic material behavior.

Nevertheless, it could be interesting on the future to identify the specific material parameters for the nonlinear damage model from the experimental curves using a two phases sequential identification procedure, as it is proposed by (Brancherie and Ibrahimbegovic 2009, Kucerova *et al.* 2009): In the first phase of the procedure, it is necessary to classify the material parameters in three identification sets:

i. elastic parameters (E, ε_{d0});

ii. compression parameters (f'_c, α_c, A_c, B_c); and

iii. tension parameters (f_t, α_t, A_t, B_t),

and define three objective functions, one for each parameter set. In the second phase, it is possible to apply an optimization method -based on an artificial neural network and an evolutionary algorithm- to these three objective functions, in order to reduce the expensive evaluation of each numerical simulation. For a more detailed description of the proposed identification procedure, see the references mentioned previously.

2.2.1 Concrete behavior: the nonlinear damage model of Mazars

The model of Mazars (1986) was specifically conceived for the non-symmetric behavior of concrete, which is different in compression compared to traction. As any other model of damage, this model is formulated to represent the loss of continuity when multiple cracks appear and grow inside the concrete. In other words, the damage in a Representative Elementary Volume (REV) corresponds to a superficial density of micro-defects that can be expressed by the equation

$$\boldsymbol{D} = \frac{\boldsymbol{S_D}}{\boldsymbol{S}} \tag{1}$$

In which $\boldsymbol{S}$ is a transversal surface without any damage, $\boldsymbol{S_D}$ is the effective surface of transfer of stresses and efforts, and $\boldsymbol{D}$ is the relationship between both surfaces, and in other words, the scalar variable of damage which goes from a value of zero (undamaged material) to one (full damage). In the case of the Mazars model, it is based on the calculation of an effective stress (Eq. (2)) which is function of two scalar damage variables, D_c and D_t-traction and compression damage respectively- (Eqs. (3) and (4)). Nevertheless, instead of building the surface of failure in the space of stresses, this one is built in the space of strains, needing the calculation of an equivalent strain (Eq. (5)).

$$\sigma = (1 - D)E^e : \varepsilon^e \tag{2}$$

$$D = \alpha_t D_t + \alpha_c D_c \tag{3}$$

$$D_i(\tilde{\varepsilon}) = 1 - \frac{(1 - A_i)\varepsilon_{d0}}{\tilde{\varepsilon}} - \frac{A_i}{\exp\left[B_i(\tilde{\varepsilon} - \varepsilon_{d0})\right]} \qquad (i = t, c) \tag{4}$$

$$\tilde{\varepsilon} = \sqrt{\sum_i \left(\varepsilon_i^+\right)^2} \qquad \varepsilon_i^+ = \max(0, \varepsilon_i) \tag{5}$$

In the last equations, α_t, α_c, A_i, B_i, ε_{d0} are model parameters that can be determined from experimental tests; the values adopted in this research are presented in Table 2, and the stress-strain relationship derived of the model is shown in Fig. 3.

2.2.2 Steel behavior: a classical nonlinear plasticity model with hardening

For the reinforcing steel bars, a classical elasto-plastic

Table 2 Material parameters for the damage model of Mazars

A_c	1.446
B_c	1570
ε_{do}	7.428E-05
A_t	0.97
B_t	8000
$f'c$ (*PSI*)	81
Confinement index	1.06
f_t (*PSI*)	407.49

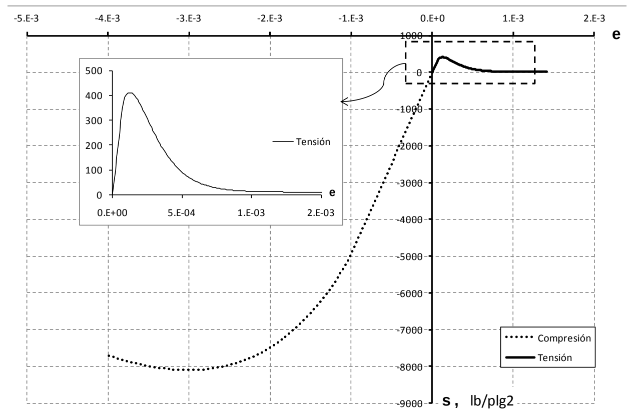


Fig. 3 Stress-strain curve for concrete behavior based on the damage model of Mazars

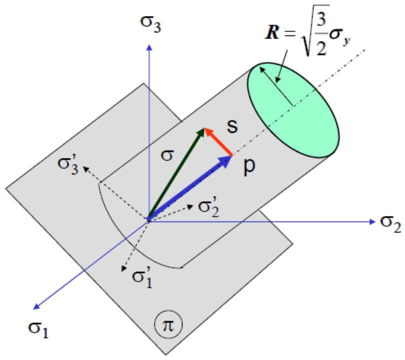


Fig. 4 Yield surface for steel behavior based on the Von Mises Criterion

model based on Von Mises Criterion was chosen, which includes isotropic hardening. According to this criterion, plasticity does not start while

$$\sigma_{eq} \leq \sigma_y \tag{6}$$

Being σ_{eq} the equivalent stress of Von Mises, which it is calculated with the expression

$$\sigma_{eq} = \sqrt{\tfrac{1}{2}\left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\right]} \tag{7}$$

In Fig. 4, it can be observed the classical plastic yield surface based on Von Mises Criterion:

For a detailed description of classical plasticity models, it can be consulted specialized bibliography (Chakrabarty 2006, Ibrahimbegovic *et al.* 1998, Ibrahimbegovic 2009).

3. Numerical analysis of the beam-column connection

3.1 Description of the numerical strategy adopted for the analysis of the B-C connection

The numerical simulations of the Beam-Column connection were made in the finite element code FEAP v.7.4 (Taylor 2005), an open-source code with license in which is possible to implement user material models and user finite elements. For this research, the 3D damage model of Mazars was specifically implemented into the code, but due to the limitations of computational memory capacity, the beam-column connection was finally modeled in a 2D-space, which it is able to provide acceptable results –in comparison with 3D models- if some simplifications are made. For example, in a real RC structural element, the steel reinforcement forms a cage embedded into the concrete, inducing a particular concentration of stresses in the concrete around each bar; however, taking into account that bending is acting only in one plane, and assuming that the most important shear stresses might be developed in the same plane, it is possible to "homogenize" the steel reinforcement in layers for a 2D simulation. Nevertheless, some careful considerations must be done in order to reproduce the real three dimensional connection's stiffness as well as the concrete's ultimate strength in a two-dimensional model: in order to obtain an equivalent two-dimensional stiffness, it was necessary to use a plane strain formulation for the finite element model, and calculate a transformed transversal section for the concrete body to accomplish with both stiffness and ultimate strength reported on the experimental results. In the case of the stirrups, only the branches parallel to the bending plane are taken into account, modeled with one truss element whose transversal section corresponds to the total area of the stirrups. Due to the autonomous construction of each finite element stiffness, it is possible to assign a particular transversal section for concrete and for steel stirrups. Because the concrete cannot develop large rotations, any possible geometrical non-linearity was not considered into the model.

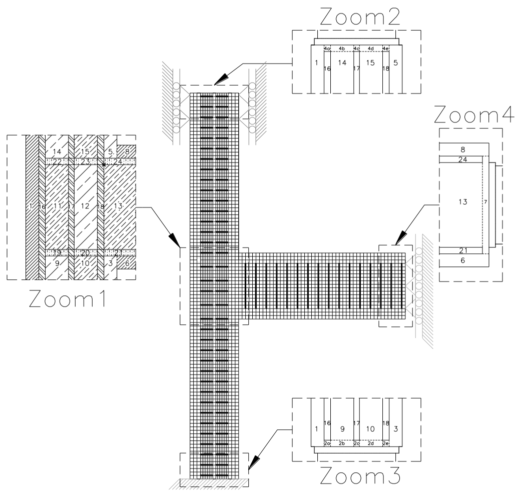
The strategy followed in this research is described in the next steps:

a) Selection of the experimental reference
b) Definition of the cases to simulate:
   ◦ only longitudinal steel without stirrups;
   ◦ with minimal quantity of stirrups;
   ◦ with the quantity of stirrups indicated in experimental test,
c) Comparison of results

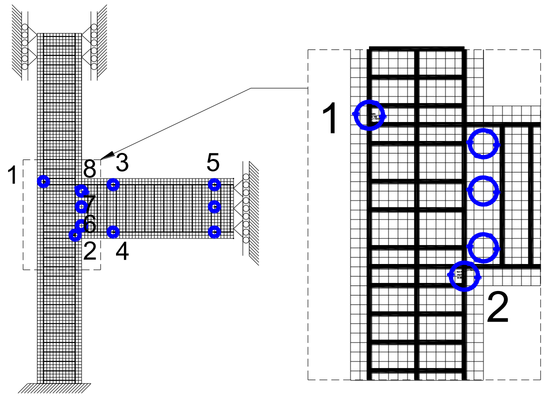
3.2 Construction of the numerical model

Based onto the proposed strategy described before, the LL11 and LH11 specimens were selected among the 12 corner-reinforced concrete beam-column subassemblies reported in the experimental reference, having the same geometrical and material properties, except for the number of stirrups inside the core of concrete (four stirrups for "Low confinement", and six stirrups for "High confinement").

The basic model was constructed in a 2D space based on a plane strain formulation, using QUAD4 elements (4-node quadrangular element with 4 integration points) for the concrete body and TRUSS2 elements (2-node bar element) for the steel reinforcement. Initially, the reinforcement was modeled with QUAD4 elements as well, but due to their minimal dimensions, there were some numerical problems by a non-realistic excessive concentration of stresses around the union between longitudinal steel and the stirrups. In which concerns to the bonding, it was modeled as perfect:



(a) boundary conditions and reinforcement array



(b) points of observation of the stress-strain relationship according to the experimental tests

Fig. 5 Meshing of the beam-column connection

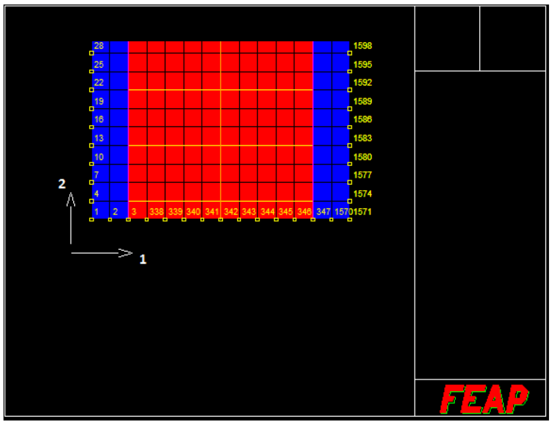


Fig. 6 Definition of the model's two concrete materials: confined (in red) and unconfined (in blue) regions

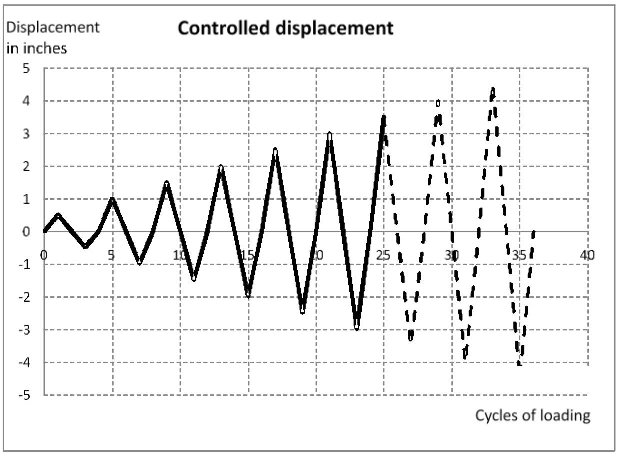


Fig. 7 Controlled displacements applied on the numerical simulations

in other words, the steel bar nodes are directly linked to the concrete body nodes. About boundary conditions, the bottom face of the column is fully-restrained, while the top face of the column was constrained only in the transversal direction because a constant axial load was applied and distributed at the same face. By the way, the free edge of the beam is restrained in the axial direction, with a cyclic displacement imposed in its transversal direction (see Fig. 5(a)). In the experimental test, at least eight displacement transducers were positioned in each Beam-Column connection to follow the evolution of displacements over the concrete face of the joint (see Fig. 5(b)). In the same way, we followed the numerical evolution of these points, in order to construct the corresponding load-displacement response.

By the way, in order to include the confinement effect induced by the stirrups and longitudinal reinforcement to the internal concrete, two kinds of concrete behavior were introduced into the model, defining two regions delimited by the longitudinal bars: internal confined concrete and unconfined external concrete (see both regions on a section of the model in Fig. 6).

In which concerns to the cycling loading, it was applied a cyclic controlled displacement with an initial value of ± ½ inches (13 mm), being increased in ± ½ inches (13 mm) in each cycle of loading, as it is shown in figure 7. However, in contrast to the experimental tests in which the specimens were subjected to nine cycles of loading, in the simulations only six cycles of loading were analyzed due to problems of convergence, associated to the total damage of concrete in some finite elements.

4. Discussion of the results

By comparison with the experimental results, we will discuss the numerical results obtained in the simulations, in which the coupling of the nonlinear behavior of steel and concrete was considered. First of all, it must be highlighted that in experimental models, beam and column elements were over reinforced in order to concentrate the damage only inside the joint. Unfortunately, in numerical simulations it is not enough to increase only the transversal area of steel rebars to reproduce the confinement effects of the over-reinforcement on concrete, so it is necessary to deal with the boundary conditions as well as with the concrete parameters in some regions of the mesh in order to avoid that damage appears immediately in some specific elements of the beam, out of the joint and near of the boundaries. As it is shown in figure 8, without these

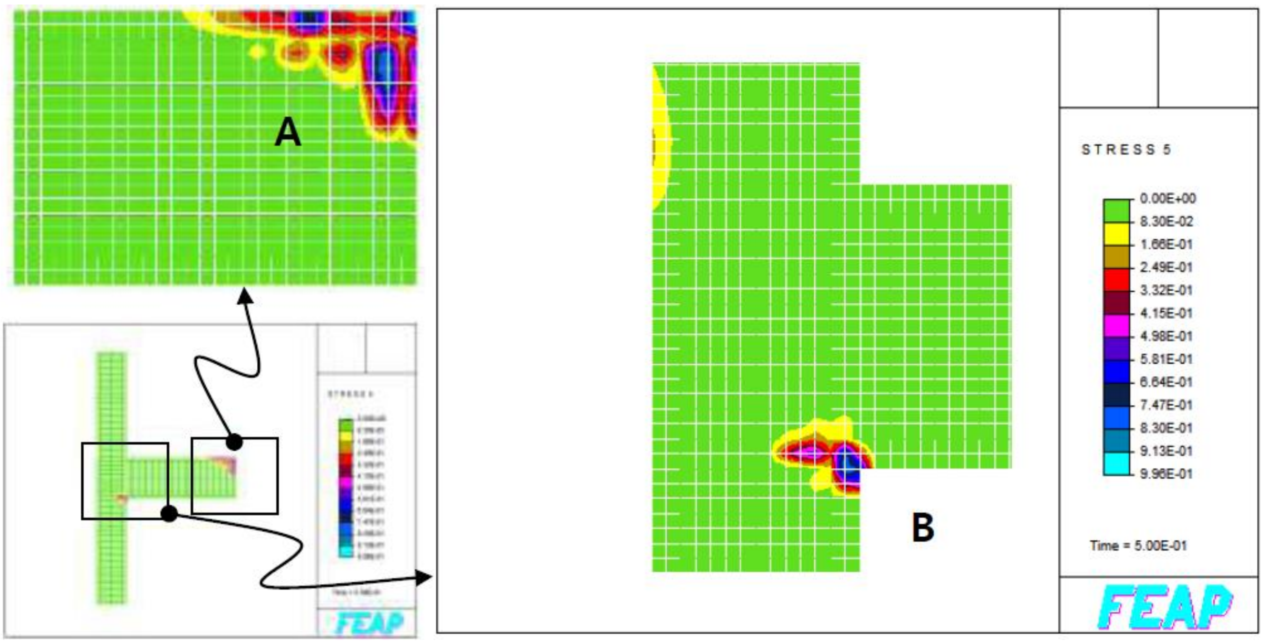
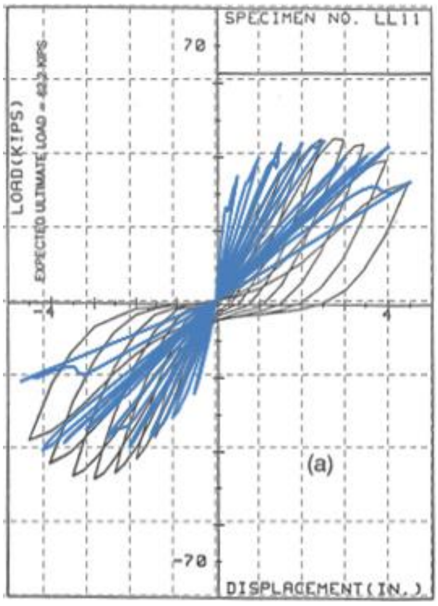
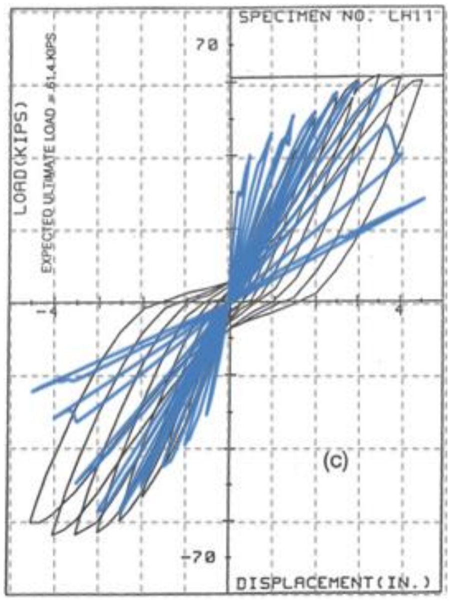


Fig. 8 First damage in the RC connection during the first cycle of loading: Premature damage in region A, and expected damage in region B



(a) Comparison of numerical and experimental curves for specimen LL11



(b) Comparison of numerical and experimental curves for specimen LH11

Fig. 9 Load-displacement structural response of the beam-column connection

modifications, the numerical damage can occur immediately in some unexpected areas of the structural joint: in this figure, letter A indicates the region with a premature damage near to the boundaries of the beam; letter B indicates the region with expected damage observed in experimental tests. This problem was solved by modifying some of the damage parameters in a region of affected finite elements, simulating the increase of concrete strength due to the confinement, and delaying the damage in this zone.

Once the inconvenient was solved, the discussion of results should focus in the comparison of the structural response of the two specimens, both experimental and numerical. Fig. 9(a) and 9(c) correspond to the LL11 specimen, in which the maximal load capacity was reached between 40 and 45 kips for a displacement near to two inches. For the LH11 specimen, Fig. 9(b) and 9(d) show a maximal load capacity near to 60 kips, very close to three inches of displacement. By comparing experimental curves with numerical results, it can be appreciated that some key-values are very similar (maximal load capacity associated to the lateral displacement), but the shape of their dissipative hysteresis loops are far away from any similitude. In numerical curves, all the unloading branches go directly to the origin, without any accumulated permanent displacement

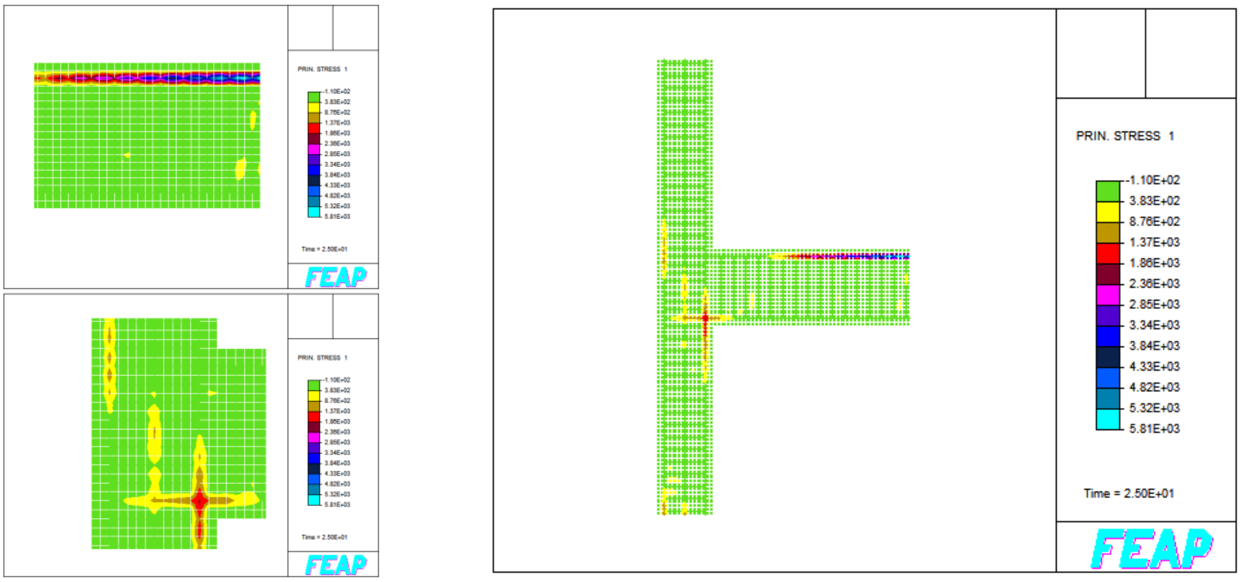


Fig. 10 Principal stress distribution on specimen LL11 (**LOW** confinement) for a displacement of 3.5 inches.

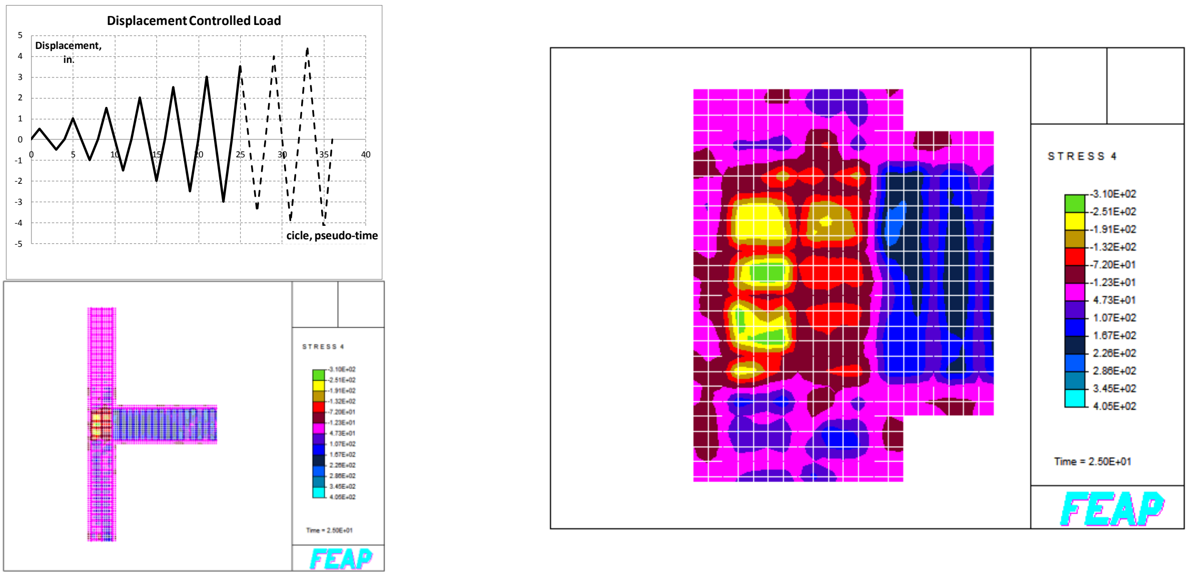


Fig. 11 Shear stress distribution on specimen LL11 (**LOW** confinement) for a displacement of 3.5 inches

as it is observed in the experiments. Typically, the origin of these permanent displacements is associated to the crack friction on concrete. For cyclic loads, the damage model of Mazars includes only the slope variation of the elastic unloading, since cracks on concrete are closed as soon as there is a reversibility of loading, assuming no friction on cracks. Because of this, it is not possible to reproduce numerically any dissipative loop or permanent deformation. This was already explained by Ragueneau *et al.* (2000), who presented a modified version of Mazars model which includes these effects. One possibility to deal with this problematic is to introduce a plastic formulation coupled to the damage model as it is proposed by Markovic *et al.* (2004), who present a coupled volume approach where the REV is linked with a fine-scale cell through the use of a multi-scale strategy. In the same line, another strategy is to adopt the visco-elastic-plastic-damage model proposed by Jehel *et al.* (2014) which is able to take into account the seismic effects on the local degradation of concrete.

The second item of discussion is the distribution of principal and shear stresses inside the specimens, as it is shown in Figs. 10, 11 and 12. In Fig. 10 it can be appreciated that principal stresses reach their maximal value along the longitudinal steel rebars, which explains the premature damage on the neighbor concrete elements, as it was discussed previously for Fig. 8. In Figs. 11 and 12, the concentration of shear stresses is determined by the disposition of the stirrups, being greater the affected area when the reinforcement is lower inside the core. In fact, when no stirrups are placed inside the core, the damage is reached almost immediately, even if the longitudinal bars of the column and beams pass through the joint. Other relevant points observed in numerical simulations are the following:

a) In both cases, the highest value of shear stress was reached on the beam, and not in the column or in the connection;

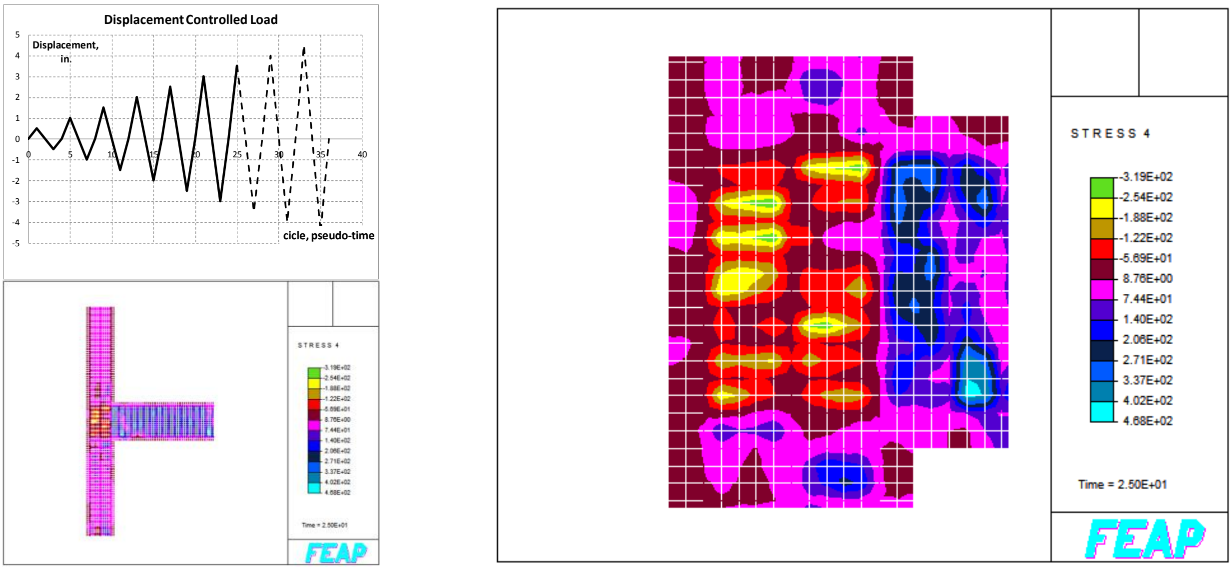


Fig. 12 Shear stress distribution on specimen LH11 (**HIGH** confinement) for a displacement of 3.5 inches

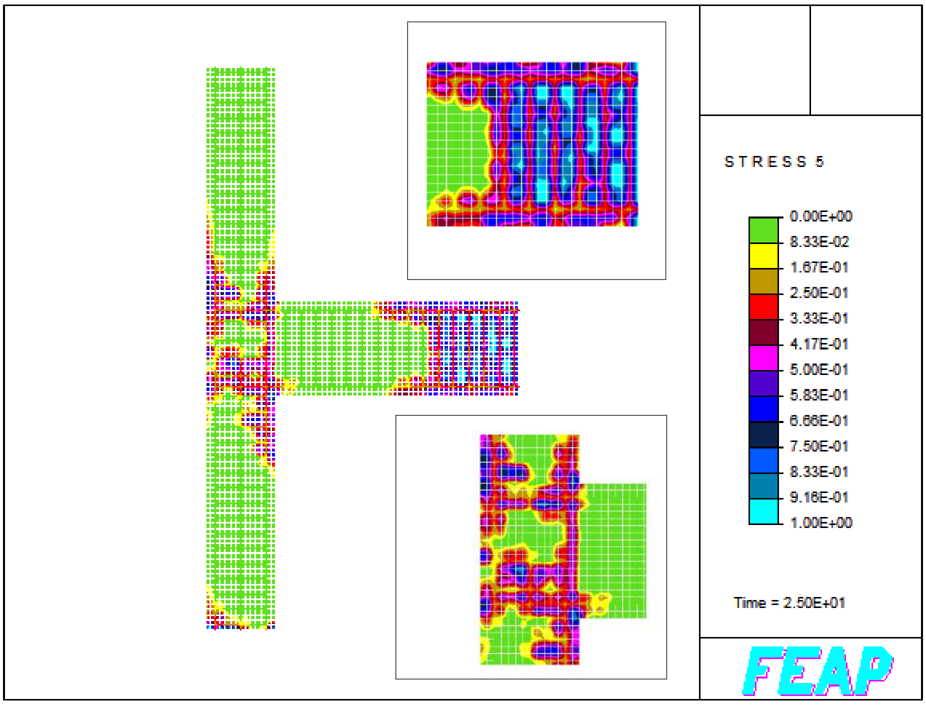


Fig. 13 Damage distribution on specimen LL11 (**LOW** confinement) for a displacement of 3.5 inches

b) When the number of stirrups is increased inside the core, the principal damage is placed out of the core, exactly in the plane of connectivity between the beam and the core of the connection (as it is observed in figure 12); and

c) If the constant axial load on the column is not included into the model, the resistance of the beam-column connection decreases substantially (which agrees with Park *et al*. 1997)).

In general, all the numerical simulations stopped as soon as a non-convergence condition was reached. Sometimes this problem was solved by reducing the time step, in particular in the picks of the displacement when unloading started. From a physical point of view, this non- convergence corresponds to the instant when a set of concrete elements reaches a high level of damage, and it is interesting to compare this numerical condition with a comment of the experimental researchers (Alamedinne *et al.* 1991), who mentioned in their work that the applied loading was very severe, in particular the final cycles of loading which are very unlikely to be experience by any real structure. In Figs. 13 and 14, the level and distribution of damage in concrete can be observed for both specimens respectively. Apparently, damage is higher in LH11 specimen, but in fact its response is more efficient than LL11 specimen's response, because the damage is better distributed along the stirrups, although the numerical value seems to be elevated. The implementation of bond elements must reduce this

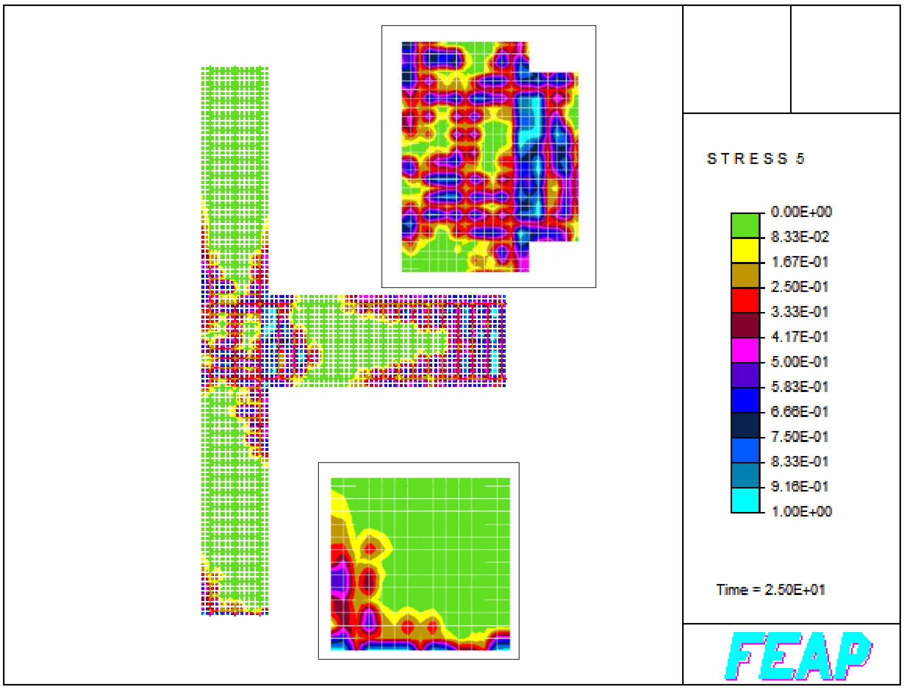


Fig. 14 Damage distribution on specimen LH11 (**HIGH** confinement) for a displacement of 3.5 inches

effect on the concrete body, as it was demonstrated by Dominguez *et al*. (2005), due to the redistribution of stresses induced by bonding, which allows a small slip or a small decohesion between steel bars and concrete, avoiding a false premature degradation of concrete as it is observed in these simulations.

5. Conclusions

The aim of this work is to demonstrate that numerical nonlinear analysis can help to better understand the structural behavior of any RC element. Considering that most of the construction codes are mainly based on experimental campaigns whereby not all phenomenological material variables are measured and registered, it is possible to enhance numerically the knowledge of the damage distribution in hidden zones, as well as improve the arrangement of the steel reinforcement inside the beam-column connection. This work focused in studying the effect of the quantity and array of the shear steel reinforcement on the structural response of a RC beam-column connection, when it is subjected to cyclic loading. The simulations were performed combining two different thermodynamics nonlinear material models: a damage model for concrete, and a classical plasticity model for steel. From a set of 12 corner-reinforced concrete beam-column subassemblies reported in the experimental reference, two specimens (the LL11 and the LH11) were modeled in order to verify the nonlinear capabilities of the numerical models for reproducing the concentration of shear stresses inside the joints. Since a numerical point of view, these simulations have served to verify the compatibility between different nonlinear formulations, but also for identifying their limitations.

The numerical results have allowed corroborating the influence of the stirrups in the resistance of the connection showing their importance, because when no stirrups are placed inside the core, the damage is reached almost immediately, even if the longitudinal bars of the column and beams pass through the joint; conversely, when the number of stirrups is increased inside the core, the principal damage is placed out of the core, exactly in the plane of connectivity between the beam and the core of the connection. Finally, it is necessary to implement bond elements which must redistribute the stresses on the concrete body if any small slip or decohesion occurs between steel bars and concrete, avoiding a false premature degradation of concrete. Nevertheless, in order to extend the influence of bonding degradation on dynamics, we require before to solve the mechanical coupling between slip and decohesion on bonding, which becomes a key point in cyclic loading/unloading, and afterwards, it will be necessary to include the corresponding dissipative effects on the structural damping, as it is suggested in Ibrahimbegovic *et al*. (2014), Jehel *et al*. (2014).

In addition to including non-perfect bonding, the numerical prediction of the cyclic load-displacement response of the beam-column connection could be improved if the concrete damage model used currently is replaced by a most robust material model: one alternative is introducing a plastic formulation coupled to the concrete damage model as it is proposed by (Markovic *et al*. 2004), who present a coupled volume approach where the REV is linked with a fine-scale cell through the use of a multi-scale strategy: the heterogeneities of concrete inside the core of the beam-column connection can play an important role in the prediction of the connection's cyclic response. A second alternative, that could include the seismic effects on the local degradation of concrete, it is developing a three-dimensional visco-elastic-plastic-damage model based on the proposal of Jehel *et al*. (2010), which takes into account the non-symmetric loading rate-dependent behavior with appearance of permanent deformations and local hysteresis, as well as other important characteristics of concrete that become relevant for earthquake engineering applications.

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