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THE EFFECT OF BOND-SLIP IN THE NUMERICAL ASSESSMENT OF RC FRAMES UNDER CYCLIC LOADING

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Abstract

Bond-slip may have significant influence on the assessment, with numerical models, of reinforced concrete structures subjected to cyclic loadings, whether static or dynamic. Its influence is discussed with the correlation of experimental and analytical results, where two numerical models are considered, including a perfect bond fiber-section finite element formulation and a force-based fiber-section model including bond-slip in the vicinity of the frame joints, both exterior and interior. In this case, the model implemented makes it possible to consider the coupling effect of the response of the beams adjacent to the joint, and the models are constructed from the geometrical and material characteristics of the structure, without any calibration procedures. The experimental results are from a reinforced concrete column and a beam-column subassemblage, both subjected to static cyclic loadings, with predefined displacements sequences for the element-ends, and from a shaking table test of a one bay two-story reinforced concrete frame structure.

The comparisons show that the considered bond-slip based model makes it possible to satisfactorily predict the response of reinforced concrete frames under both static and dynamic cyclic loadings. The influence of reinforcement slippage was evaluated by comparison of the previous results with those obtained with the perfect bond-based model. This made it possible to conclude that the accuracy of the model considering bond-slip is significantly superior to that of the perfect bond model. Furthermore, responses obtained with the previous model show the pinching effect, which is characteristic of reinforced concrete structures and significantly changes the hysteretic dissipated energy, not delivered by the latter model. This can also be seen in the effect of bond-slip in the response of the fibers which model the reinforcing rebars.

Keywords: bond-slip, reinforced concrete, cyclic loading, numerical models

1. Introduction

Bond-slip effect is known to influence the response of reinforced concrete (RC) structures, especially when subjected to intense actions such as earthquake events. Nevertheless, perfect bond based formulations are frequently used to model the behavior of these structures. In order to account for bond-slip, formulations including nonlinear springs at the finite element (FE) end nodes [1] have been proposed. These models require calibration and do not make it possible to consider the coupling effects on continuous beam-columns. Other concentrated plasticity models, with the same limitations, have been proposed [2]. Models composed of different subelements describing each relevant deformation mechanism are also available [3]. These models have the same limitations of the previous type of models and do not account for the variation of the axial force due to overturning on the hysteretic behavior. Solid models [4] are precise but highly demanding from a computational stand point.
Fiber-section models may result in an interesting compromise between accuracy and computational efficiency. An example of these models is the model by Monti and Spacone [5] which integrates the rebar element with continuous bond of Monti et al. [6] into the fiber-section force-based model proposed by Spacone et al. [7]. Even though this model is accurate and computationally efficient, it only models bond-slip in the vicinity of exterior joints. Another example of fiber-section based model with bond-slip is the one by Limkatanyu and Spacone [8], which models the coupling effect with the implementation of a rigid-panel joint element. In this model bond-slip is evaluated in the elements end nodes, thus discretization may result in a relevant number of FEs, increasing the computational demand. Based on the model by Monti and Spacone [5], a fiber-section RC beam-column model, force-based, which models the behavior of the rebars in the vicinity of exterior joints, has been proposed [9]. This model may be used together with the formulation by Monti and Spacone [5] for the assessment of RC structures, in which case the bond-slip influenced response of rebars in both exterior and interior beam-column joints is considered, maintaining the favorable characteristics of fiber-section models.

Regarding bond-slip based models validation and assessment, Monti and Spacone [5] used the results of an experimental test on a circular RC column subjected to static cyclic actions for model validation and concluded that the results obtained with their model correlate sufficiently well with the experimental data. The superiority of this model when compared with a perfect bond based model was also shown. Limkatanyu [10] used the experimental results of a rectangular column subjected to static actions for model validation. The same formulation was used in the assessment of a RC beam-column subassemblage with asymmetric beam rebars [8]. The conclusions which may be inferred from the reported results are favorable to the bond-slip model, with good correlation with the experimental results. Concerning dynamic actions, D'Ambrisi and Filippou [3] used their model to evaluate the correlation of numerical and shaking table results of a RC frame, with satisfactorily accurate predictions, with significantly better correlation with the experimental results than with a perfect bond model. Limkatanyu and Spacone [8] reported the results of study similar to the assessment by D'Ambrisi and Filippou [3], using the numerical model previously proposed [8]. The authors concluded that the effect of bond-slip was of relevance, and very good correlation with experimental results was reported.

In this paper, RC structures subjected to both static and dynamic cyclic actions are assessed, using a perfect bond formulation [7] and a bond-slip based model [9]. The latter model does not require calibration procedures, with the model parameters being defined solely by the geometric and material characteristics of the assessed structure. Moreover, this model is expected to result in a good compromise between required computational effort and accuracy.

2. Numerical models adopted

In this research, a model with perfect bond and another formulation with steel rebars bond-slip were considered. The first model is the well know force-based fiber-section beam-column FE [7]. The second model is a combination of a fiber-section FE with bond-slip in exterior joints [5] with a similar natured FE with bond-slip in interior joints [9]. Together, these two models make it possible to simulate RC frame structures, accounting for nonlinear geometric and material behavior.

The FE model by Monti and Spacone [5] models the interior joint regions behavior using the reinforcing bar element anchored in concrete with continuous bond proposed by Monti et al. [6] coupled with a length of the rebar of the beam-column outside the joint region, L_{IP}, initially determined according to the quadrature weight of the Gauss-Lobatto integration point of the end-node of the beam-column which is modelled. A length equal to the plastic hinge length of the beam-column was used [2, 9], in which case this length may be determined according to the equation proposed by Bae and Bayrak [11].

The constitutive law of the anchorage plus the rebar length inside the beam-column is given by $(E/L_{IP}\cdot k_a)/(E/L_{IP}+k_a)\Delta u_{s+a}=\Delta\sigma_{s+a}$. E is the Young modulus of steel, k_a is obtained by static condensation of the stiffness matrix of the anchorage, $u_{s+a}=\varepsilon_{s+a}\cdot L_{IP}$ is the displacement in the end of the rebar element outside the joint, ε_{s+a} is the strain of the fiber corresponding to the rebar under analysis and σ_{s+a} is the stress of this rebar. This constitutive law for the anchored rebar is used in the state determination of the fiber-section FE as the material model of the rebars of the joint control-section.

The fiber-section FE model with bond-slip in interior joints is obtained in a similar manner as the previously addressed model. In this case, the formulation models the anchorage coupled with the rebar lengths on each side of the joint, determined as indicated above. The resulting law is given by $\mathbf{k}_{12}\mathbf{k}_a/(\mathbf{k}_{12}+\mathbf{k}_a)\Delta\mathbf{u}_{s+a}=\Delta\boldsymbol{\sigma}_{s+a}$, where bold symbols represent matrices. $\mathbf{k}_a$ is obtained by static condensation of the stiffness matrix of the anchorage, now with two independent degrees of freedom, corresponding to the displacements in both extremities of the anchorage, and $\mathbf{k}_{12}$ is a diagonal matrix with terms related to the axial stiffness of the elements outside the joint, namely the corresponding values of E/L_{IP}. The two displacements in $\Delta\mathbf{u}_{s+a}$, corresponding to the displacements in both ends of the rebar elements outside the joint, are obtained as for the model of Monti and Spacone [5].

This formulation results in a three-node FE composed of two force-based fiber-section FEs linked by a node that models the interior joint. The rebars at the control-sections on each side of the interior joint, determined following the Gauss-Lobatto integration scheme, are modelled by the constitutive relation indicated above. The state determination of the control-sections of the three-node FE is the same as for the common two-node FE, but an exception is made for the control-sections of the interior joint, which are coupled because of the adoption of the previously addressed continuous anchored bar constitutive law, which depends on the response of both cross-sections. As a result, the state determination of these two sections must be performed simultaneously, as a consequence of the consideration of the coupling effect of adjacent beam-column elements. The element state determination may be computed with a residual displacements based method such as the Spacone et al. [7] method.

The constitutive relations indicated above were implemented in a MATLAB developing language based program, used in the analyses presented herein.

3. Comparison of numerical results with perfect bond and with bond-slip

3.1 RC column subjected to static cyclic loading

The first assessment to be presented is of a RC column. In this case, the Monti and Spacone [5] model, with the modification addressed above concerning the length of the rebar of the beam-column outside the joint region, was adopted. The correlation of results with both perfect bond and bond-slip based models is discussed by comparison with the experimental results of the Specimen 1 of Low and Moehle [12]. This specimen, which is a RC column, is schematically represented in Fig. 1.

The concrete compressive strength was determined as f'_{co} =36.5 MPa for unconfined concrete and f'_{cc}=42.1 MPa for confined concrete. Nevertheless, given the little difference between these values and the small thickness of the cover concrete, for simplicity, the value of the confined concrete compressive strength was considered for the entire section. The corner bars have a yielding strength of σ_y=447.5 MPa and the four bars inside the interior hoops have σ_y=504 MPa. For the two bars in the middle of the section σ_y=444 MPa. The modulus of elasticity of all the bars is E=200 GPa and the strain hardening ratio was taken as 0.8%.

Four control-sections were considered, according to the Gauss-Lobatto scheme, and, given the force-based formulation adopted, only a beam-column FE was used. The concrete cross-sections were discretized using 15 fibers, according to the corresponding Gauss-Lobatto integration points.

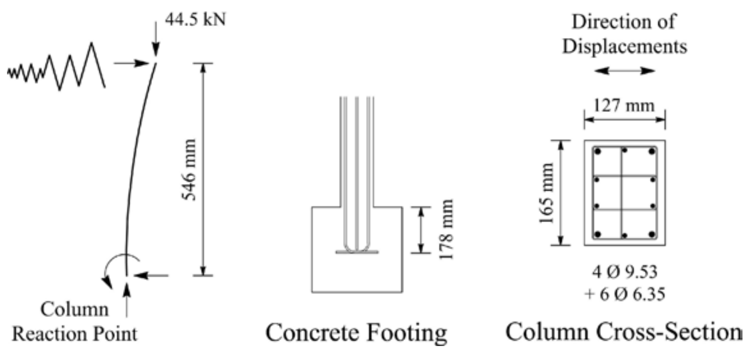

Fig. 1 – Geometry of the Specimen 1 of Low and Moehle [12]

The material models considered were the Monti and Nuti [13] for steel and the Mander et al. [14] model, with the modifications proposed by Martínez-Rueda and Elnashai [15], for concrete. For bond-slip, the model proposed by Eligehausen et al. [16] was used. All anchorages were modelled with a length of 0.178 m, in which a portion corresponding to 4ϕ is used to model the hook, following the recommendations by Eligehausen et al. [17], and a plastic hinge length of 0.25x0.127=0.032 m, determined according to the proposal by Bae and Bayrak [11], was considered. The relevant values of the bond stress-slip law were determined according to the empirical equations developed by Monti et al. [18] and adjusted as recommended by Eligehausen et al. [17] for the hook nodes. The anchorages were modelled with five FEs, including the hook, each with four Gauss-Lobatto integration points.

A perfect bond model was also used for assessment, which is the same as the bond-slip model without the anchorage behavior. This enables a more direct perception of the benefits of the consideration of yield penetration and bond-slip in the vicinity of the anchorage. The results of this analysis, in terms of column tip horizontal force as a function of the column tip displacements, are presented in Fig. 2.

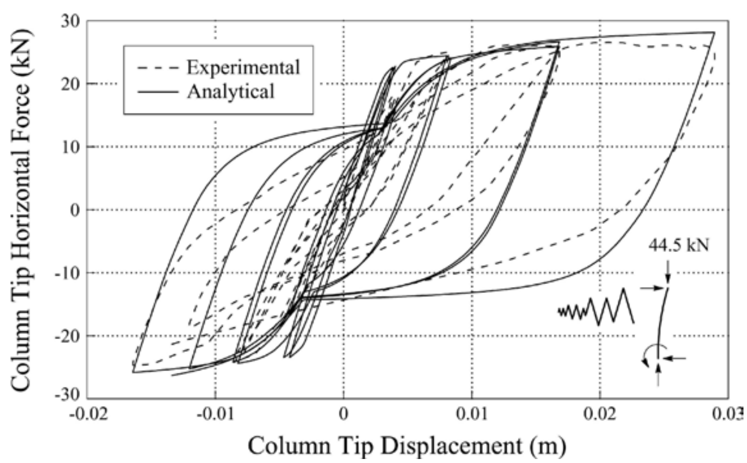

Fig. 2 – Experimental and analytical with perfect bond response of the Specimen 1 of Low and Moehle [12]

The response is fairly approximated, with the major difference lying in the overestimation of the energy dissipation. A significantly more accurate response is obtained when the anchorage behavior is modelled (see Fig. 3). A more pinched response was obtained. The dissipated energy is notably more accurate than the full bond based model prediction and the overall behavior correlates very well with the experimental results.

Even though the response of the anchorage model does not correspond to a material constitutive relation but to the response of the rebar coupled with the anchorage element (no rebar cross-section of the anchorage plus rebar fiber subsystem is subjected to the level of strain presented, which includes the anchorage slip), the origin of the differences in the responses with perfect bond and with bond-slip may be assessed by comparison between the responses of the cross-section fibers near the footing, shown in Fig. 4 for a chosen group of rebars.

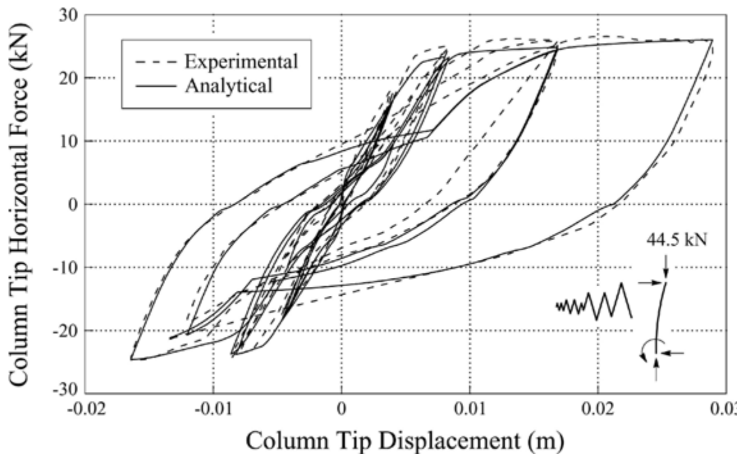

Fig. 3 – Experimental and analytical with bond-slip response of the Specimen 1 of Low and Moehle [12]

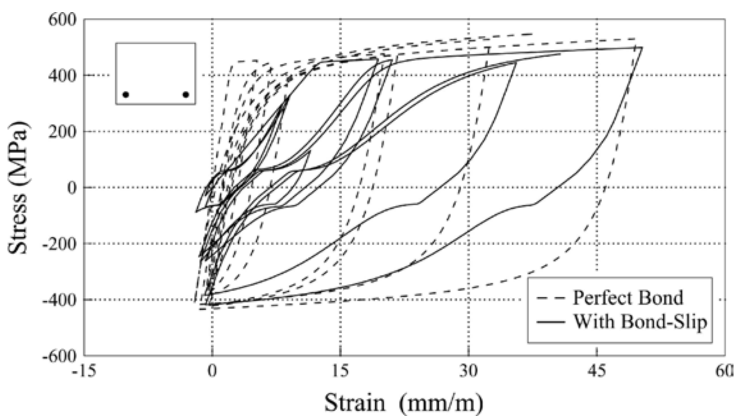

Fig. 4 – Response of the indicated fibers of the model of the Specimen 1 of Low and Moehle [12]

3.2 RC beam-column subassemblage subjected to static cyclic loading

In order to assess the response of a RC structure with an interior joint, the Specimen B13 of Beckingsale et al. [19] was considered. It represents a beam-column subassemblage, schematically represented in Fig. 5.

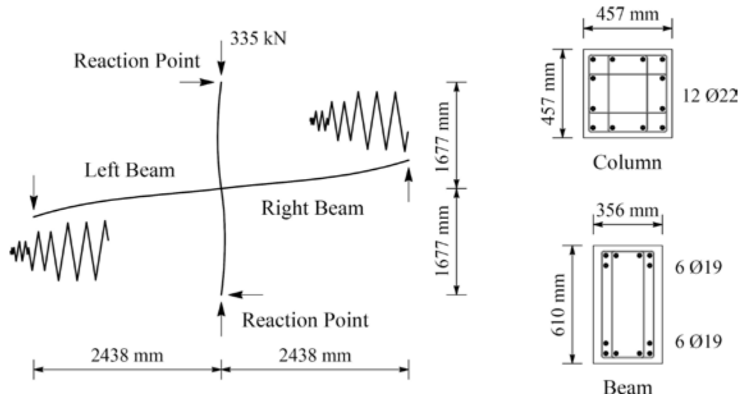

Fig. 5 – Geometry of the Specimen B13 of Beckingsale et al. [19]

The bars of the beam have σ_y=298 MPa and the rebars of the column have diameter σ_y=423 MPa. The modulus of elasticity of all the bars is E=200 GPa. For concrete f'_{co}=36 MPa was considered and the confined concrete compressive strength values were computed according to the formulas proposed by Mander et al. [14]. The confined concrete compressive strength was considered throughout the cross-sections. The material models considered were the same of the assessment of the previous section.

One beam-column FE was used for both columns, below and above the joint, with four control-sections each, and the beam was modelled with a three-node fiber-section FE, with four control-sections on each side of the interior joint. The concrete cross-sections of the columns and of the beams were defined by 20 and 25 fibers, respectively, according to the Gauss-Lobatto quadrature. For the bond-slip model, the anchorages of the interior joint of the beam were modelled with four FEs, each with four Gauss-Lobatto integration points. The length of the rebars of the beam outside the node region were determined as 0.25x0.61=0.15 m. As can be inferred from the experimental results, the slip of the anchored rebars of the column is not relevant. The material models adopted are the same indicated for the previous analysis.

The analytical response with perfect bond is depicted in Fig. 6. Similar differences between analytical and experimental results as found for the structure with an exterior joint were obtained for this structure. The correlation of these results in terms of peak force for each cycle is, as expected, satisfactory, but the prediction of the energy dissipation, with significant influence on the dynamic response of structures, results in a clear overestimation of that response parameter.

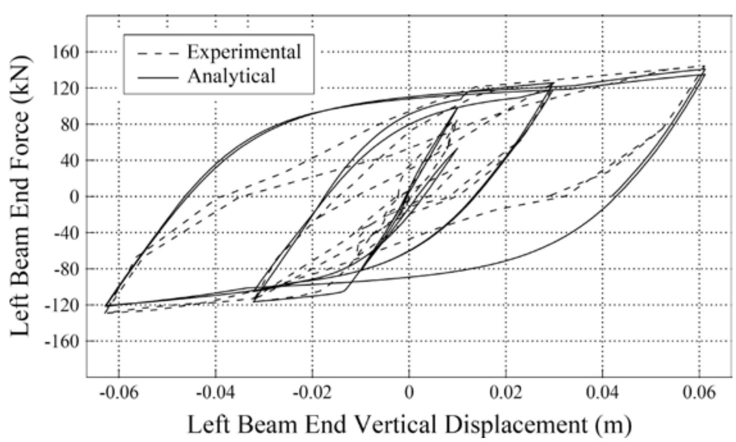

Fig. 6 – Experimental and analytical with perfect bond response of Specimen B13 of Beckingsale et al. [19]

The same comparison, considering the bond-slip of the beam rebars in the vicinity of the interior joint is presented in Fig. 7. Similarly to the results of the structure discussed above, the response with the bond-slip model is significantly more accurate, with good correlation with the experimental results. Regarding energy dissipation prediction, it is clear that the bond-slip model is significantly more accurate. This indicates that this formulation is more indicated for dynamic analysis of RC structures.

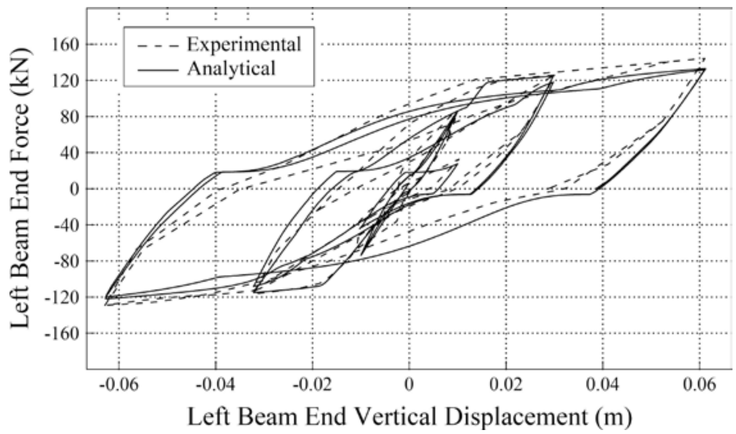

Fig. 7 – Experimental and analytical with bond-slip response of Specimen B13 of Beckingsale et al. [19]

The results for the Specimen B13 of Beckingsale et al. [19] are plotted for the maximum displacement of about 0.06 m, a high displacement for the considered structure. For the higher displacements of 0.09 m the accuracy decreases. This could have been mitigated by implementing a more complex cyclic degradation law for the bond-slip behavior [5]. Also, the consideration of the confined concrete response for the recover concrete may have contributed to this result.

3.3 RC frame structure subjected to dynamic loading

For the comparison of the results with and without bond-slip of a general RC structure under dynamic loadings, the results of the shaking table tests of a RC frame by Clough and Gidwani [20] were considered. A scheme of the geometry and rebars detailing of the frame is presented in Fig. 8. The reported actual earthquake simulated table accelerogram was used in this study. Concrete blocks were added with 35.6 kN on the top-story and to 71.2 kN on the bottom-story. The stabilization of the direction perpendicular to the frame plan was guaranteed by the construction of two connected identical frames. Two-node FEs were modelled with four control-sections and three-node FEs were defined with four control-sections on each side of the interior joint. The concrete cross-sections were discretized with 10 integration points for the columns. For the flange and web of the beams, five and 10 fibers, respectively, were used.

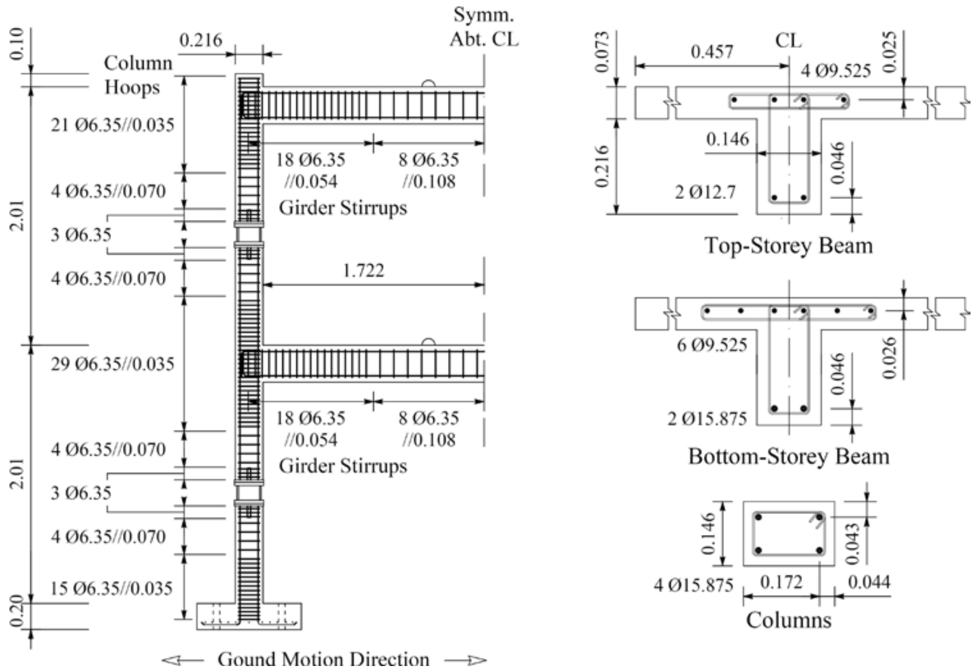

Fig. 8 – Geometry and rebars scheme of the RC frame by Clough and Gidwani [20] (dimensions in meters)

The characteristics of the materials were reported by Clough and Gidwani [20]. Regarding Steel, the most relevant characteristics are presented in Table 1, in which σ_u and ε_u are the ultimate stress and strain, respectively. For concrete the average value f'_{co}=30.3 MPa and the corresponding strain ε_{co}=3.35‰ were reported.

Table 1 – Characteristics of the reinforcement steel

ϕ (mm)	E (GPa)	σ_y (MPa)	σ_u (MPa)	ε_u (‰)
9.525	195.8	358.5	500.6	193
12.7	193.1	386.8	578.5	140
15.875	205.5	286.1	499.9	113

The model used for the assessment of the structure is schematically represented in Fig. 9. The squares represent the nodes of the model. Elements 1, 2, 5, 7, 8, 9, 10 and 12 are two-node force-based FEs with the bond-slip formulation for the exterior joint. Elements 3 and 4 are three-node force-based with the formulation addressed above, and elements 6 and 11 are fiber-section FEs with perfect bond. The bottom column ends were considered to be fixed, in line with the test conditions. The weight of the elements and of the concrete blocks were lumped at the nodes. Pinned linear elastic elements, with high stiffness, were implemented, making it possible to consider the concrete block masses at their real position. Two-node FEs were modelled with four control-sections, and three-node FEs were defined with four control-sections on each side of the interior joint. The concrete cross-sections were discretized with 10 integration points for the columns, and five control points for the flange and 10 fibers for the web in the case of the beams.

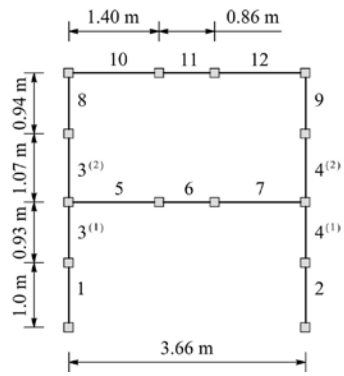

Fig. 9 – Discretization and geometry of the model of the RC frame by Clough and Gidwani [20]

The material models adopted are the same indicated for the tests discussed above. Because the nonlinear behavior is considered directly by the material constitutive models adopted, and because viscous damping is usually irrelevant when compared with hysteretic damping, Rayleigh damping was assumed with ζ=0.1% for the first two natural vibration modes.

To solve the equations of motion, the Hilber, Hughes and Taylor [21] α-method was used with α=-0.05. A time step of 0.005 s was used.

The comparison between the experimental results and the analytical responses with perfect bond and with bond-slip is depicted in Fig. 10 for the bottom-story and in Fig. 11 for the top-story.

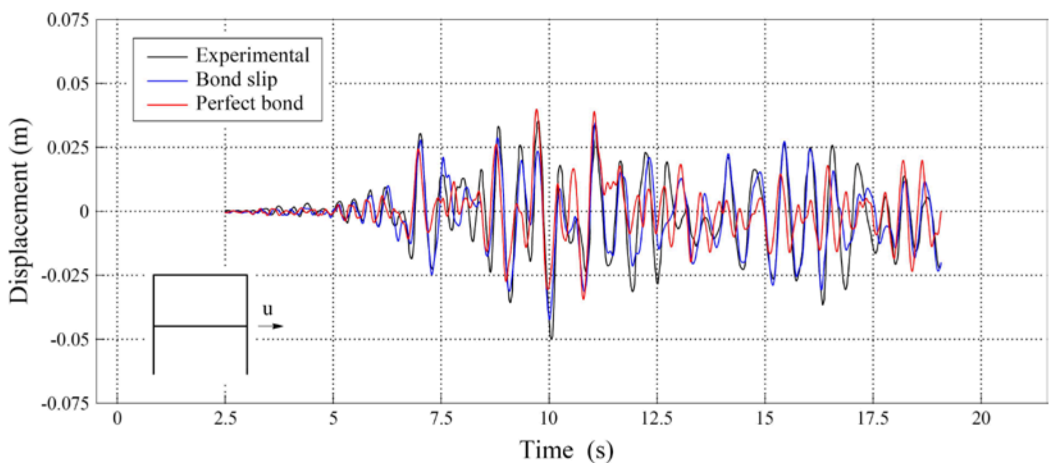

Fig. 10 – Comparison of the bottom-story displacement responses of the RC frame by Clough and Gidwani [20]

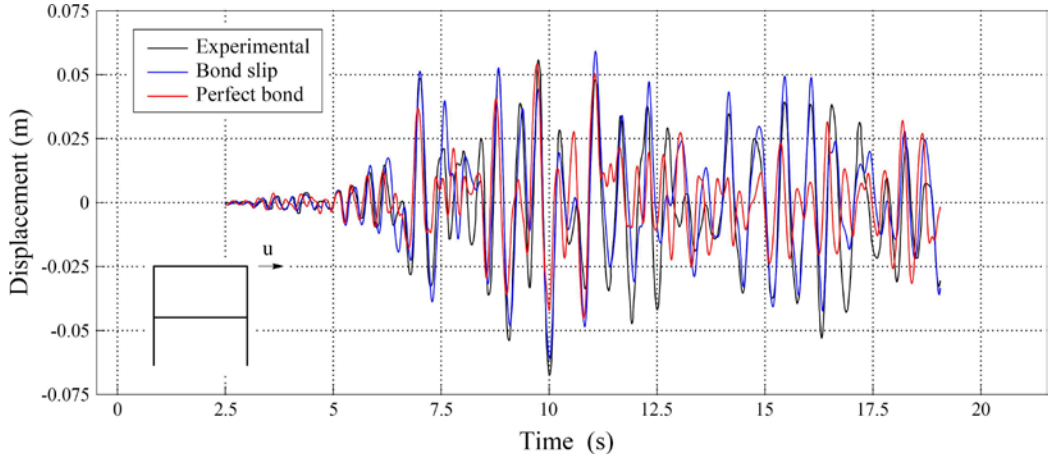

Fig. 11 – Comparison of the top-story displacement responses of the RC frame by Clough and Gidwani [20]

The response obtained with the model with bond-slip is very satisfactory, with good correlation with the experimental results. This correlation is significantly superior to that obtained with the perfect bond based model. For further assessment, the fiber responses with and without bond-slip of a chosen group of rebars of a control-section of an interior joint are depicted in Fig. 12 and Fig. 13.

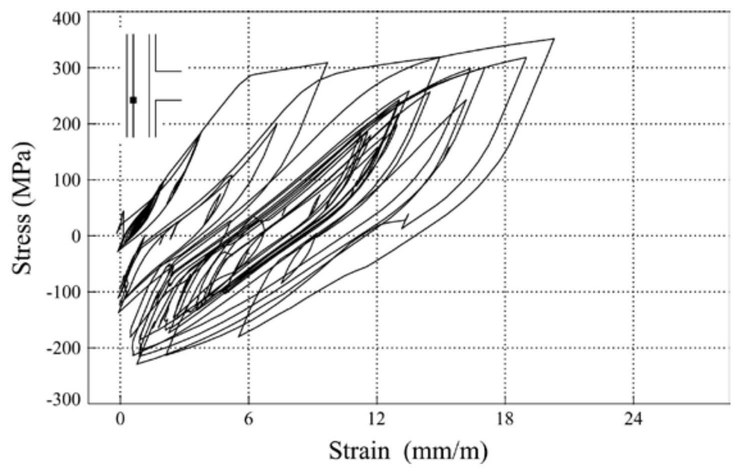

Fig. 12 – Response of a rebar fiber of the model with bond-slip of the RC frame by Clough and Gidwani [20]

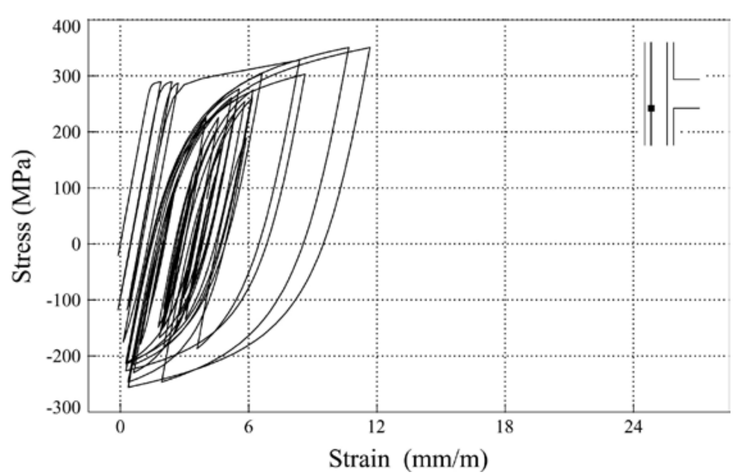

Fig. 13 – Response of a rebar fiber of the model with perfect bond of the RC frame by Clough and Gidwani [20]

4. Conclusion

In this paper the influence of bond-slip in the prediction of the response of reinforced concrete structures using numerical models is discussed. Two numerical models were adopted, namely a perfect bond fiber-section FE model and a fiber-section model including bond-slip in the vicinity of the frame joints. The evaluation is made by comparison of results obtained with these two models with experimental results from a reinforced concrete column and a beam-column subassemblage, both subjected to static cyclic loadings, with predefined displacements sequences for the element-ends, and from a shaking table test of a one bay two-story reinforced concrete frame structure.

Even though more complex models exist, the results of the fiber-section force-based models with bond-slip adopted are very satisfactory, what makes it possible to conclude that these formulations constitute a good compromise between accuracy and computational efficiency. It should be noted that the used models were constructed solely by the geometric and material characteristics of the assessed structure, not requiring any *ad-hoc* calibration procedures. For all the assessed cases, the results obtained with perfect bond based formulations delivered significantly more accurate predictions of the response. The results of the analyses with bond-slip are similar to the experimental results and more pinched responses, when compared with the perfect bond formulation results, were obtained. This can also be seen in the effect of bond-slip in the response of the fibers which model the reinforcing rebars in the vicinity of both exterior and interior structural joints. The dissipated energy is notably more accurate than the full bond based model prediction and the overall behavior correlates very well with the experimental results.

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