NONLINEAR ANALYSIS OF REINFORCED CONCRETE FRAMES INCLUDING BOND-SLIP EFFECTS

S. Limkatanyu[1] and E. Spacone[2]

[1] *Assistant Professor, Dept. of Civil Engineering , Faculty of Engineering, Prince of Songkla University, Songkhla, Thailand*
[2] *Professor, PRICOS Department, University of Chieti, Pescara, Italy*

Email: suchart.l@psu.ac.th, espacone@unich.it

ABSTRACT :

The slips of the reinforcing bars and deterioration of the bond interfaces above the foundations and in the beam-column connections and plastic hinge regions play a crucial role in describing the behavior of reinforced concrete frames under earthquake excitations. These effects result in the reduction of stiffness and energy dissipation, leading to the characteristic pinched hysteretic loop. This study proposes an analytical model which is simple and computational efficient but capable of representing the salient features of reinforced concrete members under cyclic loading. As a result, the nonlinear dynamic analysis of the structural system is performed with reasonable computational effort. The analytical model is comprised of frame elements and beam-column joint elements. This proposed model is verified through comparisons with experimental data, in particular an reinforced concrete column and interior reinforced concrete beam-column sub-assemblages, and a small reinforced concrete frame tested on a shaking table. The results are in good agreement with the experimental evidences and show the importance of including the bond slip effects.

KEYWORDS: Reinforced Concrete, Bond-Slip, Hysteretic Models, Cyclic Loads, Seismic Loads, Nonlinear Frame Analysis.

1. INTRODUCTION

The accurate representation of the reinforcement slippages is essential in predicting the response of reinforced concrete (RC) frames subjected to both static and dynamic loadings. Under the assumption of full composite actions between the concrete and the steel rebars, the stiffness of RC structures is overestimated, as is the hysteretic energy dissipated during cyclic loads. Experimental tests on RC subassemblages have indicated large fixed-end rotations at the structural member ends. These fixed-end rotations are resulted from slippage of the rebars passing through the joints or being anchored into the footings. Under cyclic loads, bond gradually deteriorates, and additional flexibility leads to the characteristic pinched hysteretic loops observed in several tests. The inclusion of the bond-slip effects into numerical models is a crucial step toward the development of accurate nonlinear techniques for the analysis of RC frame structures.
Major achievements in modelling the static and dynamic responses of RC structures have been accomplished in recent years, especially in the development of fiber frame models. The main advantage of the fiber-section model is that it automatically couples the axial and bending effects. However, most fiber-section frame models available in the published literature are typically based on the assumptions that plane sections remain plane and that there are full composite actions between the concrete and the steel rebars, thus neglecting the bond-slip effects. This leads to an overestimation of the initial stiffness and of the hysteretic energy of RC structures [1,2]. The most elementary way to include the bond-slip effects in frame elements is to add nonlinear springs at the member ends [3]. Although simple, this approach requires the formulation of an *ad-hoc* phenomenological moment-rotation relation, and disrupts the continuity of the fibers between adjacent elements. Monti and Spacone [4] explicitly include the bond-slip effects into the fiber section without relaxing the strain-compatibility between the concrete and the rebar. The total steel fiber strain is the sum of two components: the strain in the rebar, plus a strain equivalent to the rebar slip. The element is efficient and accurate, but rather complicated to implement because it combines the force-based element state determination [1] with the steel fiber iteration loop needed to determine the contribution of the steel strain and of the bond-slip to the total fiber strain [4,5].
In this paper, the numerical models recently developed by the authors consist of a displacement-based RC frame element with bond-slip [6] and a rigid-panel joint element with bond-slip [7]. The main objective of this paper is to use these models to investigate how bond-slip affects the cyclic response of RC structures through the study of two structural specimens (a beam-column joint and a two-story/ one-bay frame) previously tested in the lab at other research institutions. The flexibility-based fiber frame element without bond-slip proposed by Spacone et al. [1] is used to compare the accuracy of the fiber elements with and without bond-slip in assessing the response of the experimentally tested specimens.

2. NUMERICAL MODELS

2.1. Displacement-Based RC Frame Element with Bond-Slip

The 2-node displacement-based RC frame element with bond-slip used in this study is presented in Limkatanyu and Spacone [6] and is shown in Fig. 1. The RC frame element is comprised of a 2-node beam and *n* 2-node bars with bond-interfaces. The nodal degrees of freedoms of the beam and of the bars are different to allow reinforcement slippages.

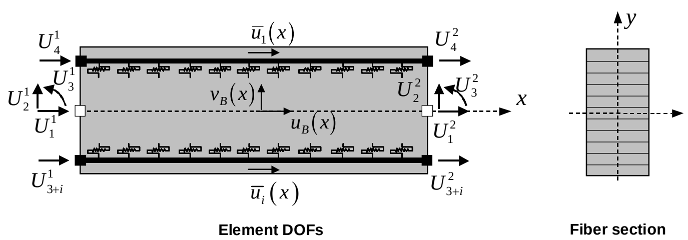

Figure 1 RC frame element with bond-slip

2.2. Plane Rigid-Panel Joint Element with Bond-Slip

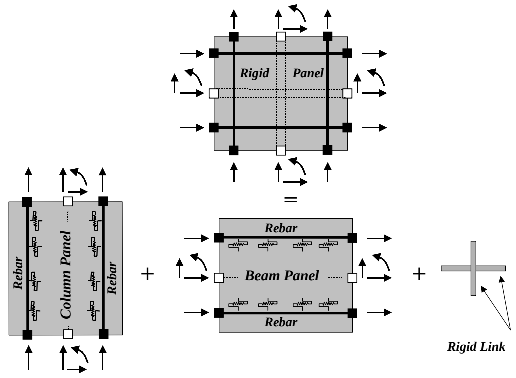

Figure 2 Rigid-panel joint element with bond-slip

Fig. 2 shows the plane rigid-panel joint element with bond-slip proposed by Limkatanyu [7]. The joint element is consisted of a beam-panel, column-panel, and a rigid-link member. These two panels are assumed to be independent of each other and are connected together through a rigid-link member to eliminate spurious rigid body modes. The same rotation is imposed at each face of the joint element. The slippage of the rebars passing through the joint is explicitly included.
The rigid-panel assumption is reasonable when the shear deformations of the joint are negligible. This assumption is substantiated by the research conducted by Pantazopoulou and Bonacci [8], who carried out the database analysis of 143 specimens of exterior and interior beam-column joints tested over a span of 35 years in the US, Canada, Japan, and New Zealand. Pantazopoulou and Bonacci [8] conclude that joints should be designed with sufficient hoop reinforcement so that their load carrying capacity exceeds that of the adjacent members. Furthermore, they observe that none of the specimens analyzed failed because of joint shear failure, even when poorly reinforced, because of the confinement provided by the transverse beams framing into the joint representing the typical configuration in internal joints. Consequently, the proposed joint element is best suited for the aforementioned situation and is intended to account mostly for bond-slip effects, while shear panel failures are not of concern in this study. All the numerical models used in this study are implemented in the general-purpose finite element program FEAP [9].

3. COMPARISON OF EXPERIMENTAL AND NUMERICAL RESULTS

Two correlation studies between experimental and numerical results are used to verify the accuracy of the models with bond-slip and to investigate the bond-slip effects on the cyclic response of RC frames. The first application considers the cyclic test of a RC beam-column subassemblage where large fixed-end rotations are observed at the column faces due to the slippage of the rebars passing through the joint. The second application considers an RC frame dynamically tested on the shaking table at the University of California, Berkeley.

3.1. Beckingsale et al.[10]: specimen B11

A series of interior RC subassemblages were tested by Beckingsale et al. [10]. These specimens were intended to represent 2/3 models of a beam-column system in a typical building in the range of 10 to 15 stories. One of these subassemblages, labelled specimen B11, is used for this study. The configuration of Specimen B11 is shown in Fig. 3. The column was subjected to a constant axial compression of 335 kN (approximately 5% $f_c^{'}A_g$) and cyclic vertical displacements at both beam-ends.

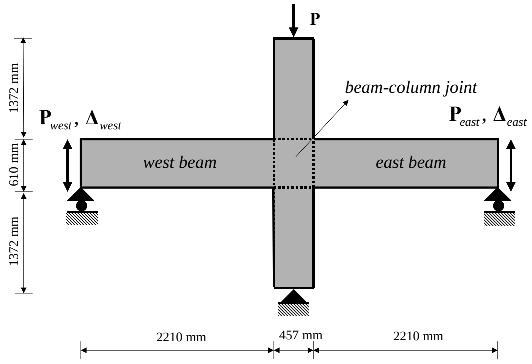

Figure 3 Geometry and loads of specimen B11

Fig. 4 (a) compares the west-end experimental response with the numerical response obtained with the model with bond-slip, while Fig. 4 (b) compares the experimental response with the numerical response obtained with the model without bond-slip proposed by Spacone et al. [1]. The Similar response is obtained for the east-end beam. As expected, both models succeed in determining the beam strength. However, the model with bond-slip gives a much better prediction in terms of stiffness and hysteretic energy dissipated. The beam–end response obtained with the model with bond-slip clearly shows the patterns of slightly spindle-shaped hysteretic loops also visible in the experimental responses.

Bond-slip mainly affects the shape of the unloading-reloading branches. During unloading, initial unloading is followed by closing of the cracks at the beam-column interface, reloading, and yielding of the steel in tension. With the model with bond-slip, when the beams unload, closing of the beam-column interface cracks is accompanied by slip of the rebars passing through the beam-column joint. Crack closure is indicated by an abrupt change of stiffness in the specimen response. This phenomenon results in a fixed-end rotation at the beam-column faces that leads to a more flexible response. The different response in the positive and negative loading directions is due to the fact that the area of top reinforcement in the beams is twice that of the bottom reinforcement. Therefore, yielding of the beam bottom rebars in tension cannot induce yielding of the beam top rebars in compression. This implies that the top beam-column interface cracks do not close when the beam-end forces induce compressive stresses in the top bars. On the other hand, the bottom beam-column interface cracks close every time the beam-end forces induce compressive stresses in the bottom bars. By the time the beam-column interface cracks open throughout the section depth, the Bauschinger effects in the bottom rebars also reduce the beam-section stiffness. The formation of cracks in the beam-column joint does not affect the integrity of the joint due to the sufficient amount of joint reinforcement, even tough the density of cracks observed during the experimental tests increased during the loading history [10]. Consequently, the contribution of the joint panel to the inelastic behaviour of the specimen is insignificant and the assumption of rigid concrete in the joint panel seems acceptable.

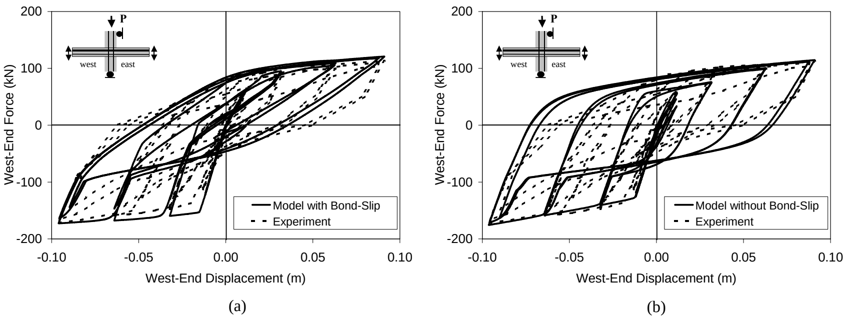

Figure 4 Experimental and numerical responses of specimen B11 (west-end beam)

Fig. 5 (a) and (b) show the stress distributions of the outer-layer top and bottom bars along the beam, respectively, at different cycles. The rebar stress distributions clearly show the push-pull phenomenon for the rebars passing through the joint. The rebars are in compression at one side and in tension at the opposite side.

This results in stress gradients much steeper along the joint than along the beams framing into the joint. This implies higher bond demands in the joint region. The yield penetration toward both beam-ends from the column faces is about 300 mm for the beam rebars. The column rebars did not yield during the analyses [7]. The same observation is reported in the discussion of the experimental results [10]. This is due to the fact that specimen B11 was designed based on a strong column-weak beam design philosophy.

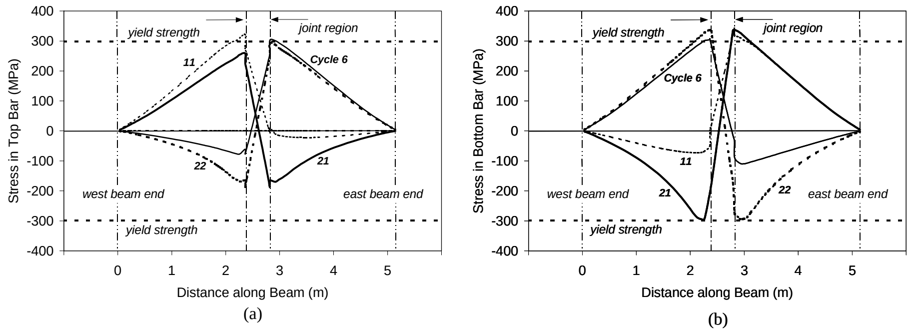

Figure 5 Steel bar stress distributions along beam for specimen B11: (a) top bar; (b) bottom bar

Fig. 6 (a) and (b) show the bond slip distributions of the outer-layer top and bottom bars along the beam, respectively. These distributions are reported for the same cycles for which the steel stresses are shown of Error! Reference source not found.. In general, the slips of the rebars inside the joint are much higher than along the beams, due to the high rebar stress gradients shown in Fig. 5. These large slips inside the joint result in large fixed-end rotations at the beam-column faces also observed in the test [10].

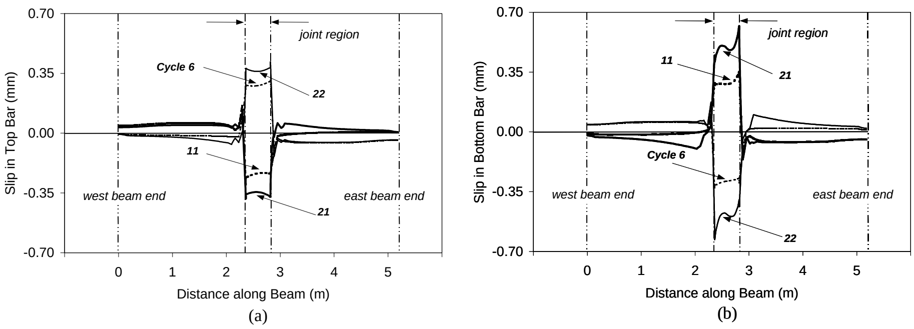

Figure 6 Steel bar bond-slip distributions along beam for specimen B11: (a) top bar; (b) bottom bar

3.2. Clough and Gidwani [11]: specimen RCF2

The correlation study with a two-story one-bay RC frame specimen tested on the shaking table at the University of California, Berkeley is used to further validate the accuracy of the frame element with bond-slip in predicting the nonlinear dynamic response of RC frames. The frame, referred to as specimen RCF2, represent a 0.7 scale model of a typical two-story office building. The details of the test specimen and of the test set-up on the shaking table are described in Clough and Gidwani [11]. During the shaking table test the specimen RCF2 was subjected to three consecutive ground motions. The characteristic of each ground motion corresponds to the N69W Taft record from the Arvin-Tahachapi earthquake of 21 July 1952. The peak ground acceleration of each ground motion was scaled to 0.095g, 0.57g, and 0.65g. The three ground motions are referred to as W1, W2, and W3, respectively. In this study, only the ground-motion W2 is of interest because the shear cracks in the girders are minimal and the beam-column joints still maintain integrity, thus implying that the behaviour of the structure is dominated by the flexure associated with yielding and slippage of the reinforcement bars. This behaviour is best suited for validating the proposed frame and joint models. Specimen RCF2 consists of two identical parallel frames supporting the girders with two applied masses, as shown in Fig. 7. The specimen is symmetric along the longitudinal axis, and the ground motion direction is applied along this axis. Using symmetry, only one plane frame is modelled. Fig. 7 also shows the dimensions of the girder and of the column cross-sections.

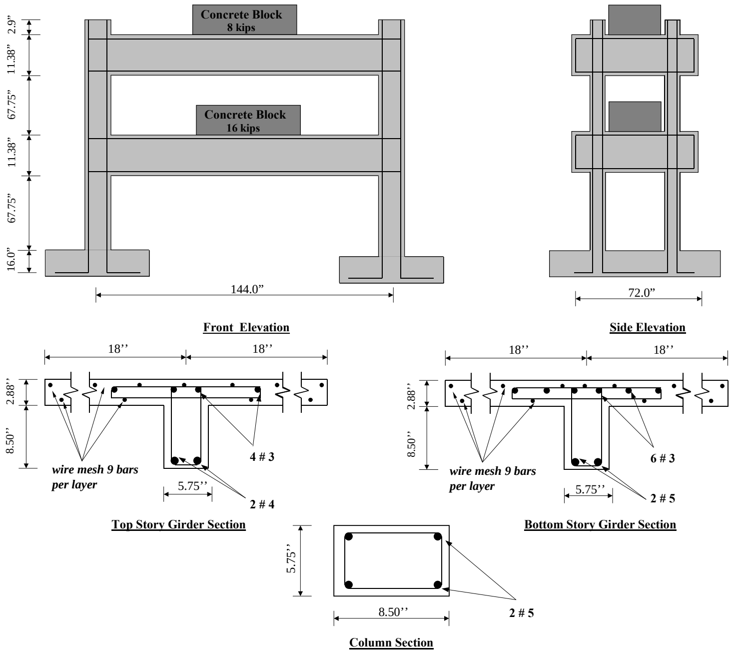

Figure 7 Girder and column cross-sections for specimen RCF2

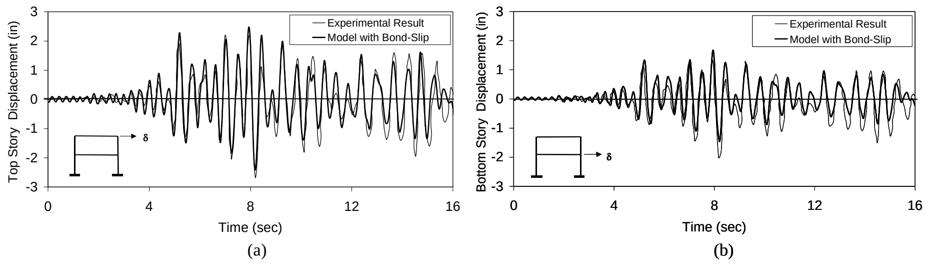

Figure 8 Experimental and analytical displacement response for specimen RCF2

Before the start of the first shaking table test W1, Clough and Gidwani [11] performed a snap test and the resulting measured natural frequencies were 3.80 and 9.80 Hz. for the first and second mode of vibration, respectively. Before the start of the second shaking table test W2, the snap test was repeated and showed natural frequencies of 3.13 and 8.70 Hz. The frequency drop was mostly due to concrete cracking and some damage following test W1 [11]. The second set of natural frequencies is used as a reference for calibrating the analytical model. The natural frequencies of the modal analysis performed on the analytical model before imposing the ground-motion W2 yields frequencies equal to 3.05 and 7.43 Hz. for the first and second mode of vibration, respectively. 3% Rayleigh damping ratios are assumed for both modes of vibration.

Fig. 8 (a) and (b) show the comparison between experimental and analytical displacement histories of the top and bottom story, respectively. From the response histories it is clear that the first mode of vibration dominates the frame response. Fig. 8 shows overall good agreement between experimental and numerical results. The model with bond-slip succeeds not only in matching the frequency content of the response and the general waveform of the response but also in predicting the maximum displacements. Fig. 9 compares the experimentally measured and the analytically predicted total base shear histories. Similarly to the conclusions drawn from Fig. 8, the numerical model with bond-slip is successful both in representing the structural frequencies and the general waveform of the response and in predicting the maximum base shear. Fig. 10 (a) plots the bottom story shear versus the corresponding drift. It shows a pattern of a slightly spindle-shaped hysteretic loop. The stiffness degradation results mainly from crack opening and concrete crushing at the column bases while the pinching is caused by slippage of the column rebars anchored in the foundations, which leads to a column base rotation. Fig. 10 (b) presents the relation between top story shear and top story drift. Fig.

10 (b) does not show yielding of the column reinforcement and the small amount of hysteretic damping is mainly caused by the hysteretic behaviour of the concrete. Pinching is still present in the response due to the slippage of the column rebars passing through the beam-column joints. No major inelasticity aside from concrete cracking was observed in the girders.

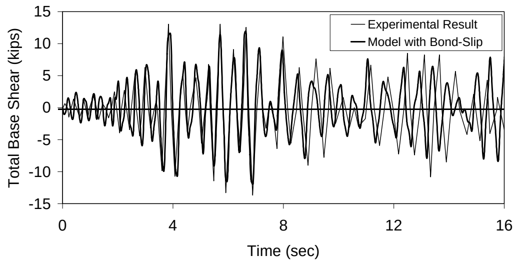

Figure 9 Experimental and analytical base shear response for specimen RCF2

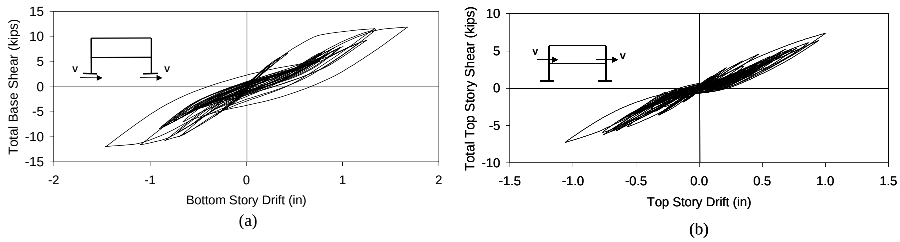

Figure 10 Shear force-interstory drift response of specimen RCF2 obtained with model with bond-slip

In order to show the importance of bond-slip in assessing the response of the frame, the analyses were repeated with the model with perfect bond proposed by Spacone et al. [1]. The concrete and steel material properties are those used in the analysis with bond-slip. The modal analysis performed before imposing the ground-motion W2 yields the natural frequency values of 3.25 and 7.85 Hz. for the first and second mode of vibration, respectively. As expected, these natural frequency values are slightly larger than those obtained from the modal analysis of the model with bond-slip, indicating a stiffer model. 3% Rayleigh damping ratios are used for the first and second mode of vibration. Fig. 11 (a) and (b) show the effects of bond-slip on the response histories of the top and bottom story displacement, respectively. For the bottom story displacement, the maximum displacement obtained with the model with bond-slip is approximately 51 % larger than that obtained with the model without bond-slip. For the top story displacement, the maximum displacement obtained with the model with bond-slip is approximately 65 % larger than that obtained with the model without bond-slip. Much of this difference derives from bond-slip in the foundations and in the beam-column joints. Fig. 11 clearly shows a stiffer response in the case of the model without bond-slip.

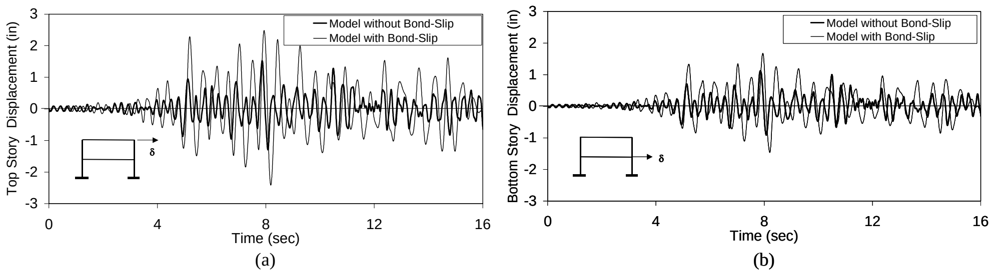

Figure 11 Effects of bond-slip on displacement response of specimen RCF2

Fig. 12 compares the base shear response histories with and without bond-slip. While the two histories are different, the influence of bond-slip on the maximum base shear force is insignificant. This happens because the base shear is related to the product of the structural lateral stiffness and of the structural interstory drift. When the effects of rebar slippage are taken into account they cause a decrease in the lateral stiffness and at the same time an increase in the lateral displacements. It seems that in this particular application these two factors cancel each other out. The same observation is also reported by D'Ambrisi and Filippou [12].

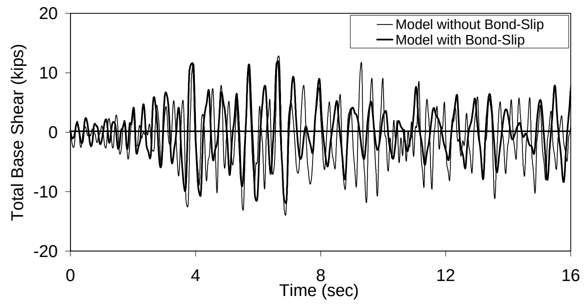

Figure 12 Effects of bond-slip on base shear response for specimen RCF2

Finally, Fig. 13 (a) and (b) show the relation between story shear and interstory drift displacements for the bottom and top story, respectively, for the analyses without bond-slip, and should be compared to Fig. 10. Similarly to the case of the model with bond slip, most of the inelastic action is concentrated at the bottom story columns while the top story response is basically elastic. Compared to the responses of Fig. 10, the pinching characteristic is not observed in Fig. 13 due to the perfect-bond assumption. Comparing Fig. 10 (a) and Fig. 13 (a), the hysteretic-energy dissipation in the case of the model without bond-slip is much more pronounced than in the case of the model with bond-slip. This is due to the column base rotation caused by the rebars slippage in the foundations.

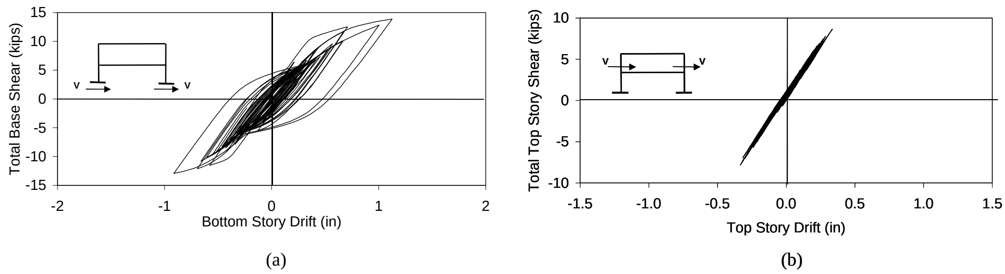

Figure 13 Shear force-interstory drift response of specimen RCF2 obtained with model without bond-slip

4. CONCLUSIONS

The applications of two recently developed RC models that explicitly account for the bond-slippage of the steel rebars are presented in this study. Two correlation studies of two experimentally tested RC structural specimens (a beam-column joint and a two-story frame) are used to verify the model accuracy and to investigate the effects of reinforcement slippages.

In the first application, the study of the RC beam-column subassemblage validates the model accuracy and shows how including the effects of bond-slip leads to a slightly spindle-shaped hysteretic loop and to a good assessment of the amount of hysteretic-energy dissipation while excluding the bond-slip effects overestimates the amount of hysteretic-energy dissipation. The bond-slip effects do not affect the loading capacity of the structure since no slip failure was observed in either the experimental test of the analytical results. The stress and bond-stress distributions of the rebars indicate that the bond demand along the joint is critical and that the rebar slippage inside the joint results in large fixed-end rotations at the beam-joint interface. In this application the assumption of rigid concrete in the joint-panel is reasonable due to the sufficient joint reinforcement provided.

In the second application, the study of the shaking-table test of a two-story one-bay RC frame, very good

correlation is observed when bond-slip is included, while the model with perfect bond leads to an overestimation of the hysteretic damping of the frame and an underestimation of the structural flexibility and of the displacement demands. Similarly to the first application, most of the slip effects are observed inside the joints and in the footings and result in fixed-end rotations that shorten the structural frequencies.
Overall, the analytical models used are quite simple and can be easily implemented in a nonlinear code and are general enough to be applied to other problems involving slip. The authors are using the same models to study the response up to failure of RC beams strengthened with Fiber Reinforced Polymers (FRP). Other applications in the field of concrete structures are the study of regions with insufficient lap-splices (a problem not uncommon in older European bridge piers built in seismic areas) and the model extension to concrete structures prestressed with both steel and FRP cables. Finally, a further enhancement of the joint element should model the effects of joint shear failure, which may be important in the study of external joints that lack the beneficial confinement of the cross beams found in interior joints.

5. ACKNOWLEDEMENTS

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