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Investigating the effect of bond slip on the seismic response of RC structures

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Abstract. It is reasonable to assume that reinforced concrete (RC) structures enter the nonlinear range of response during a severe ground motion. Numerical analysis to predict the behaviour therefore must allow for the presence of nonlinear deformations if an accurate estimate of seismic response is aimed. Among the factors contributing to inelastic deformations, the influence of the degradation of the bond slip phenomenon is important. Any rebar slip generates an additional rotation at the end regions of structural members which are not accounted for in a conventional analysis. Although these deformations could affect the seismic response of RC structures considerably, they are often neglected due to the unavailability of suitable models. In this paper, the seismic response of two types of RC structures, designed according to the Iranian concrete code (ABA) and the Iranian seismic code (2800), are evaluated using nonlinear dynamic and static analyses. The investigation is performed using nonlinear dynamic and static pushover analysis considering the deformations due to anchorage slip. The nonlinear analysis results confirm that bond slip significantly influences the seismic behavior of RC structure leading to an increase of lateral deformations by up to 30% depending on the height of building. The outcomes also identify important parameters affecting the extent of this influence.

Keywords: nonlinear dynamic analysis, nonlinear static analysis, bond slip, RC building, seismic performance

1. Introduction

When RC structures are subjected to strong ground motions, they might develop inelastic behavior as it has been confirmed in the past earthquake events. The nonlinear performance of a structure subjected to earthquake depends on its ability to tolerate large inelastic deformations in critical regions. During the past two decades, researchers have increasingly investigated the nonlinear dynamic performance of RC structures (Alsiwat *et al*. 1992, Ayoub and Filippou 1999, Limkatanyu and Spacone 2002, Oh and Kim 2007, Liu 2007, Sezen and Setzler 2008, Wang and Liu 2009, Yanchao and Zhong-Xian 2009, Dominguez *et al*. 2010, Shang *et al.* 2010). These studies have resulted in the development of many analytical and numerical methods, some of them

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very efficient, though most if not all require a high degree of care with respect to modeling in order to achieve correct results.

Amongst these studies, there are some cyclic models (Alsiwat *et al*. 1992), which have been developed in order to simulate the behavior of critical regions. However, most of these only consider the flexural response of members, while there is no single model available that allows for the nonlinear effects of shear and bond slip concurrently. In this study, nonlinear dynamic analyses of two case-study structures are performed using the analysis program Drain-RC (Drain-RC 2006, Shooshtari 1998). This program considers nonlinear deformations due to bond slip based on the model developed by Alsiwat and Saatcioglu (1992), Alsiwat *et al*. (1992). The effects of bond slip on the nonlinear behavior of the structure are then discussed.

2. Modeling of bond slip

Bond slip is a major component of inelastic deformations in RC structures. It occurs when the critical flexural section of one member is located near the joint region. Flexural cracking at the interface of two members results in reinforcement elongation. Widening of crack width leads to inelastic strains in the steel reinforcements which results in the penetration of yielding into the joint region. Additional rigid-body deformations may also occur due to the slippage of reinforcement. The combined effect of the reinforcement elongation and slip in the joint region is called the bond slip. Different models have been developed to consider the effect of bond slip in the nonlinear response of RC structures (Otani and Sozan 1972, Filippou *et al*. 1983, Morita and Kaku 1984, Alsiwat *et al*. 1992, Limkatanyu and Spacone 2002, Kwak and Kim 2006, Oh and Kim 2007, Sezen and Setzler 2008, Yanchao and Zhong-Xian 2009, Dominguez *et al*. 2010). In this research, the model of Alsiwat and Saatcioglu (1992) and has been implemented along with their well-presented laboratory results.

The models, illustrated in Fig. 1, can be defined by the following expressions:

Anchorage extension:

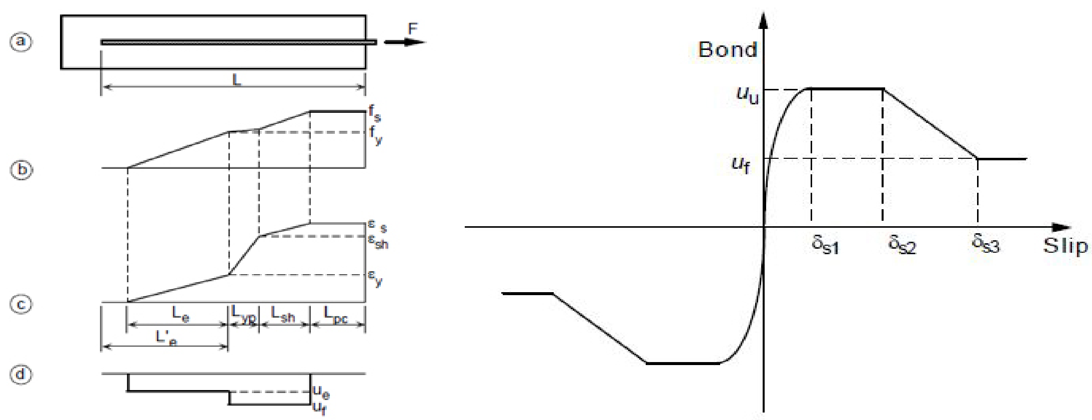


Fig. 1 Anchorage extension and bond-slip models developed by Alsiwat and Saatcioglu (1992)

$$u_e = u_{ACI} = \frac{f_y d_b}{4L_d} \quad (Mpa)\ ;\ L_d = \frac{440 A_b}{k\sqrt{f'_c}} \frac{f_y}{400} \geq 300\ \ mm\ ;\ L_e = \frac{f_s d_b}{4u_e}$$

Where u_e = the elastic bond stress (MPa); f'_c = concrete strength (MPa); f_y = steel yield strength (MPa); d_b = bar diameter (mm); L_d = development length (mm); and A_b = the bar area (mm^2); f_s = the maximum elastic steel stress (MPa). Coefficient K reflects the effects of confinement and bar spacing, and can be taken as $3d_b$.

$$u_f = \left(5.5 - 0.07\frac{S_L}{H_L}\right)\sqrt{\frac{f'_c}{27.6}} Mpa\ ;\ L_{yp} = \frac{\Delta f_s d_b}{4u_f}$$

where S_L and H_L are the clear spacing and height of lugs on the bar respectively, and Δf_s is the incremental stress between the beginning and end of the yield plateau region. The extension of bar, δ_{ext}, can be computed by integrating the strains.

$$\delta_{ext} = \varepsilon_s L_{pc} + 0.5(\varepsilon_s + \varepsilon_{sh}) L_{sh} + 0.5(\varepsilon_{sh} + \varepsilon_y) L_{yp} + 0.5(\varepsilon_y) L_e$$

Reinforcement Slip:

$$u = u_u \left(\frac{\delta_s}{\delta_{s1}}\right)^{0.4} \qquad \delta_s \leq \delta_{s1}$$

$$u = u_u \qquad \delta_{s1} \leq \delta_s \leq \delta_{s2}$$

$$u = u_u + \left(\frac{u_f - u_u}{\delta_{s3} - \delta_{s2}}\right)(\delta_s - \delta_{s2}) \qquad \delta_{s2} \leq \delta_s \leq \delta_{s3}$$

$$u = u_f \qquad \delta_s \geq \delta_{s3}$$

$$u_u = \left(20 - \frac{d_b}{4}\right)\sqrt{\frac{f'_c}{30}}$$

$$\delta_{s1} = \sqrt{\frac{30}{f'_c}}(mm)\ ;\ \delta_{s2} = 3.0(mm)\ \ \text{And}\ \ \delta_{s3} = \text{clear distance between the lugs.}$$

All stress terms in these expressions are in MPa, while slip terms are in mm.

$$\delta_s = \delta_{s1}\left(\frac{u'_e}{u_u}\right)^{2.5}$$

$$u'_e = 0 \quad \text{if} \quad L'e > Le\ ; \quad u'_e = \frac{f_s d_b}{4L'_e} \quad \text{if} \quad L'e \leq Le$$

Where u'_e = the elastic bond stress at the far end of the bar; L'_e = the available elastic length of the bar.

3. Case study buildings

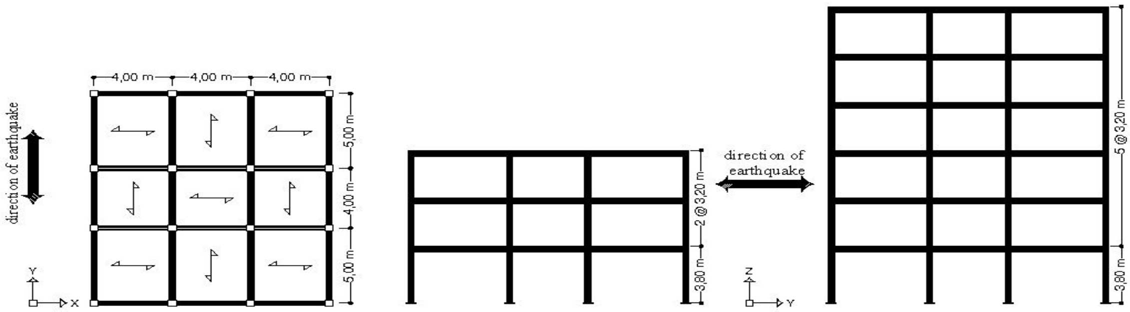


Fig. 2 The plan of case-study structures, along with the bending frames and the direction of roofs ribbing

Table 1 Reinforcement arrangements and sectional size of the 3- and 6-story buildings

Building	Floor	Column		Beam	
		Section	Reinforcement	Section	Reinforcement
3 Stories	first	350×350	8 ϕ 22	350×450	4 ϕ 22 top, 2 ϕ 22 bottom
	second	300×300	8 ϕ 20	350×400	4 ϕ 22 top, 2 ϕ 22 bottom
	third	300×300	8 ϕ 20	350×400	2 ϕ 22 top, 2 ϕ 20 bottom
6 Stories	first	450×450	12 ϕ 24	400×500	5 ϕ 25 top, 4 ϕ 25 bottom
	second	450×450	8 ϕ 24	400×500	5 ϕ 25 top, 4 ϕ 25 bottom
	third	400×400	8 ϕ 24	400×500	5 ϕ 24 top, 3 ϕ 24 bottom
	fourth	400×400	8 ϕ 24	400×500	4 ϕ 24 top, 3 ϕ 24 bottom
	fifth	300×300	8 ϕ 24	300×400	4 ϕ 22 top, 2 ϕ 22 bottom
	sixth	300×300	8 ϕ 22	300×400	2 ϕ 22 top, 2 ϕ 22 bottom

Two 3- and 6-story buildings are analyzed for this study. These buildings are classified as residential and their respective plans have similar dimensions of 12×14 m, as shown in Fig. 2. The height of the first story is 3.8m and 3.2m in all other stories.

Vertical loading is estimated according to the recommendations of the Iranian code for Loading of Buildings (2001), and seismic loading is estimated according to the Iranian seismic code (Iranian Code of Practice for Seismic Resistant Design of Buildings 1999). Seismic coefficient was determined based on the soil type III which is for silty clay soil of medium density and peak ground acceleration of 0.3g representing a high seismic hazard; the behavior factor is taken as 8 to simulate RC moment resisting frame with medium ductility; concrete compressive strength and reinforcement yield strength were selected as 30 MPa and 300 MPa, respectively. Reinforcement details and dimensions of the two buildings are provided in Table 1.

4. Numerical modeling of the structures

Nonlinear analysis of the structures was performed using Drain-RC software (Drain-RC 2006). Due_to the limitation of the software, one interior and one exterior frame were selected and linked together by a rigid (very high stiffness) element. The details are as follows:

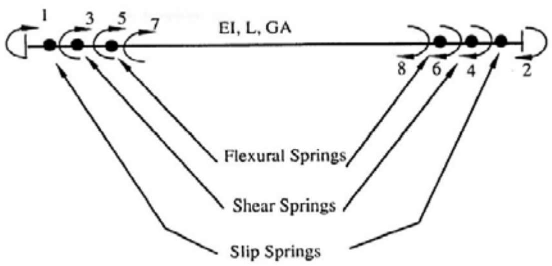


Fig. 3 Elastic element with the 3 springs; model of Alsiwat and Saatcioglu (1992)

4.1 Loading

Vertical loading included dead load plus 20% of live load while earthquake loading was based on the earthquake records of Tabas, Naghan, San Fernando and El Centro and scaled in order to reach a peak ground acceleration of 0.3g.

4.2. Beams and columns

For inelastic analysis, each member was modeled as an elastic member along with three torsion springs at each end of member, as illustrated in Fig. 3.

Flexural and slip springs were defined based on bilinear moment-curvature curves and slippage moment-rotation of the section while the shear spring was defined based on the tri-linear envelope of shear moment-rotation curve of the section (cracking, yield and post yield). In order to consider hysteretic behavior, Takeda rule was used for bending (Takeda *et al*. 1970), Ozcebe's model for shear (Ozcebe and Saaticioglu 1898), and Alsiwat rule for slippage (Alsiwat and Saatcioglu 1992). The aforementioned modeling approach has been introduced in Drain-RC program Alsiwat and Saatcioglu (1992), Alsiwat *et al*. (1992).

4.3 Determination of envelope curves for moment-curvature and moment-rotation

In order to determine the bilinear envelope of moment-curvature, the moment versus slip rotation curve and the trilinear envelope of moment versus shear rotation moment-rotation curve, firstly the moment-curvature and the moment-rotation curves were obtained for each of beams and columns using COLA software (Yalcin and Saatcioglu 1999). The required bilinear and trilinear curves were found using added sub programs to COLA. Cracking and yield points and pre- or post-yielding gradients were then obtained (Fallah 2003). Figs. 4, 5 and 6 present examples of moment-curvature and moment-rotation envelopes, respectively.

4.4 Damping

According to the Iranian seismic code (Iranian Code of Practice for Seismic Resistant Design of Buildings 1999), the structure damping was assumed to be %5 of the critical damping, and coefficients of mass and stiffness damping (β, α) were calculated and introduced to the software considering the first and the second free vibration modes.

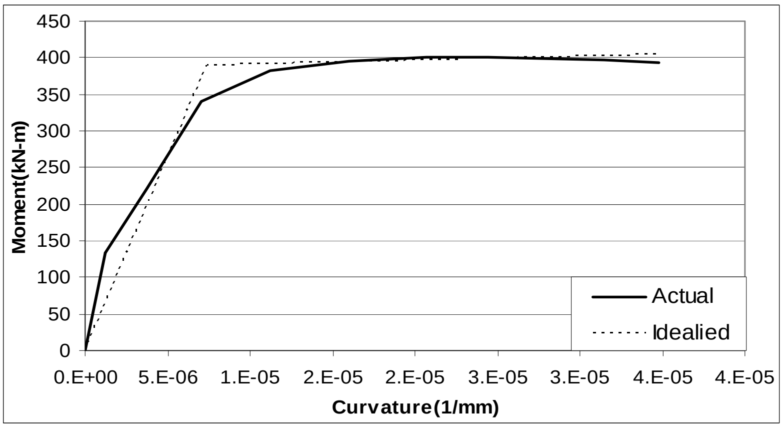


Fig. 4 Bilinear idealization of moment-curvature curve for the first floor column of the 6-story building

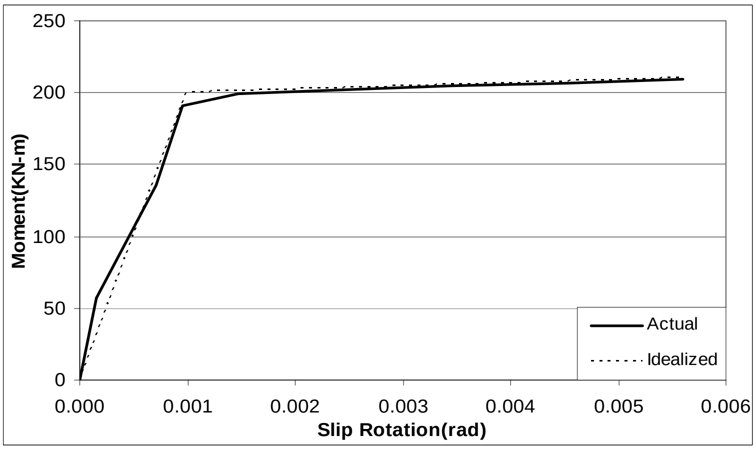


Fig. 5 Bilinear idealization of moment versus slip rotation for the second floor column of the 6-story building

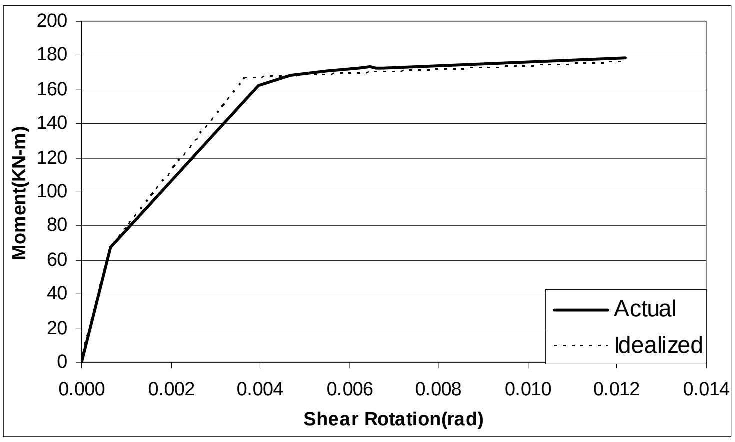


Fig. 6 Trilinear idealization of moment versus shear rotation for the fourth floor beam of the 6-story building (l = 5m).

Table 2 Analysis methods used for evaluation (Fallah 2003)

Type of analysis	Linear-Static	Nonlinear-Static	Nonlinear Dynamic			
Software	SAP 2000	Drain-RC	Drain-RC			
Loading	Method of 2800 certificate	Push Over method	Tabas record	Naghan record	ELcentro record	Sanfranando record
Analysis token	LSA	PUS	TAB	NAG	ELC	SAN

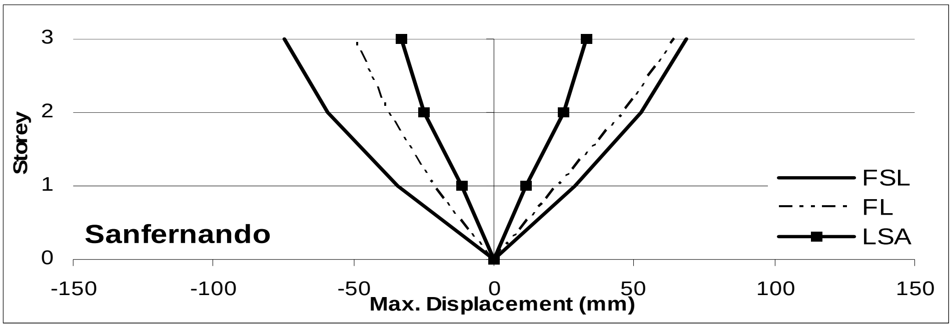


Fig. 7(a) Maximum displacement at each story of the 3-story building

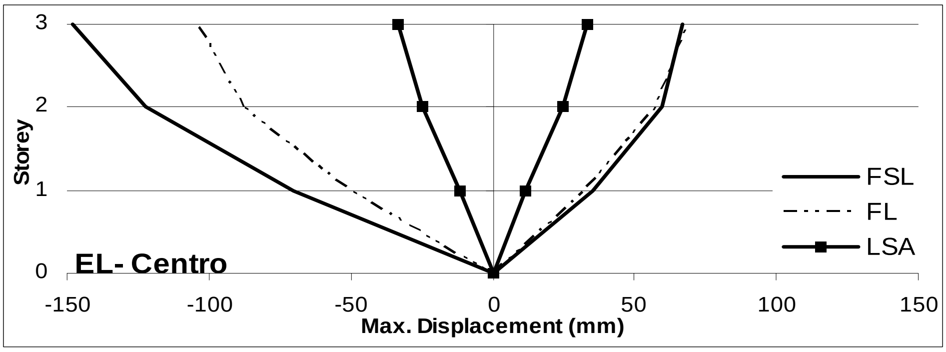


Fig. 7(b) Maximum displacement at each story of the 3-story building

5. Different states of structures evaluation

Both structures are studied for two different states: with and without the influence of bond slip:

- Structure in normal state only subjected to flexural deformation's effects. (FL)

- Structure subjected to flexural deformation's effects and bond slip non linear deformations. (FSL)

Both structures were analyzed with different methods according to the following strategy:

A) Linear static analysis with SAP2000 software under static load equivalent to earthquake, according to the Iranian seismic code provisions.

B) Nonlinear static analysis (pushover) using Drain-RC software under loading pattern and according to the Iranian seismic code provisions.

C) Nonlinear dynamic analysis by Drain-RC software with measured acceleration time history of Naghan (NAG), Tabas (TAB), Elcentro (ELC) and Sanfernando (SAN).

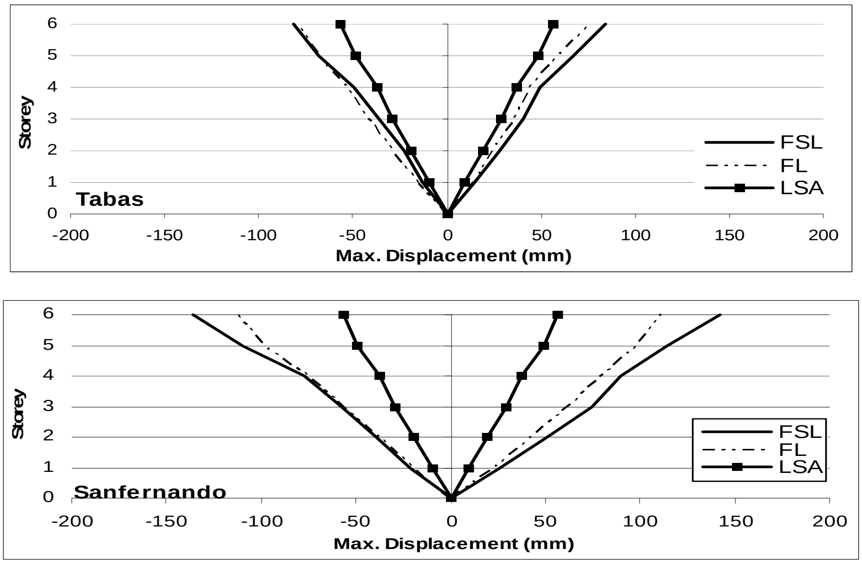


Fig. 8 Maximum displacement at each story of the 6-story building

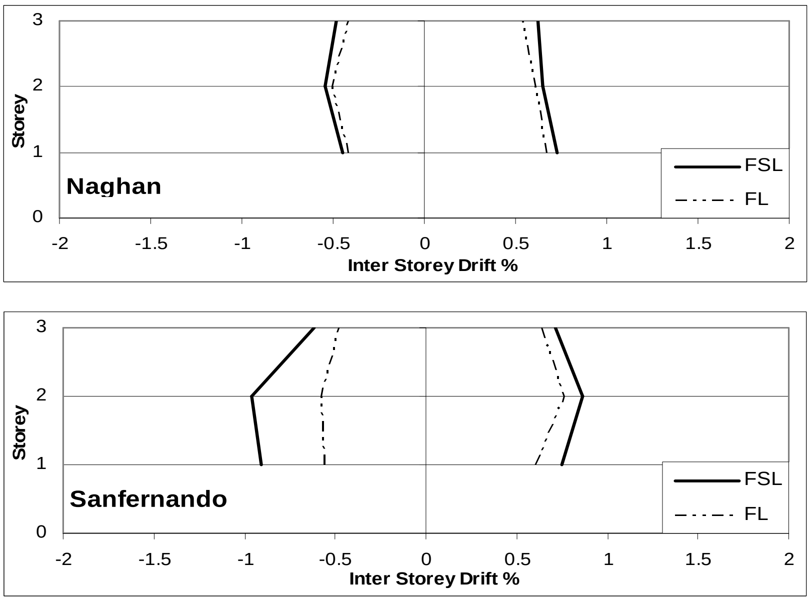


Fig. 9 Inter-story drift in the 3-story building

6. Comparing deformations of the studied structures

In Figs. 7-12, actual and relative displacements of all stories are provided for the two cases: with and without bond slip effects. As it is seen for the 6-story buildings under Sanfernando and El-Centro excitation, there is a significant difference between the two cases although the extent of this difference depends on the type of earthquake and the height of structure. A summary of this comparison is provided in Table 3.

Fig. 10(a) Inter-story drift in the 6-story building

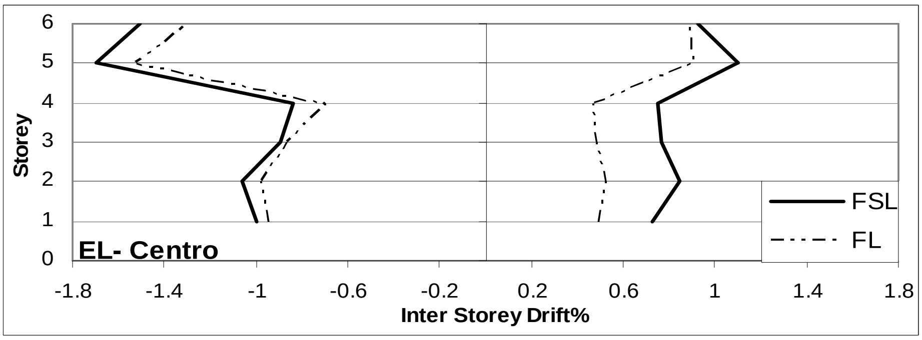
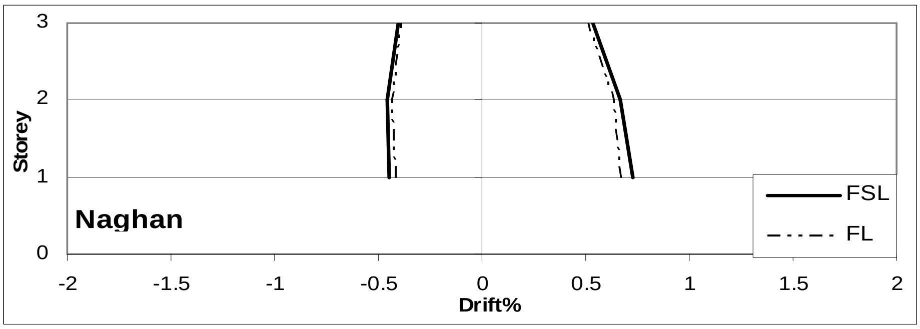


Fig. 10(b) Inter-story drift in the 6-story building



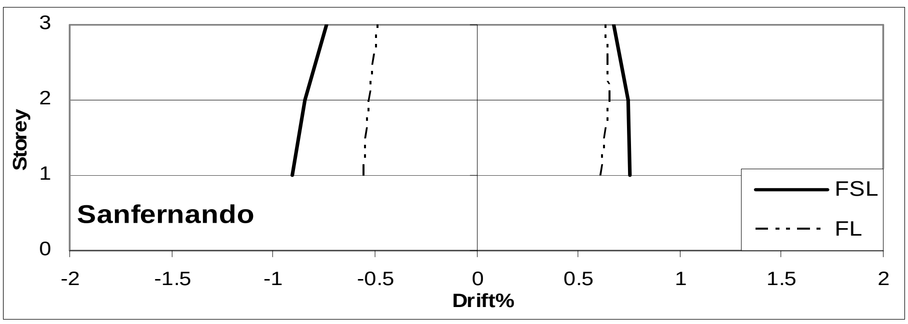


Fig. 11 Drift at each story for the 3-story building

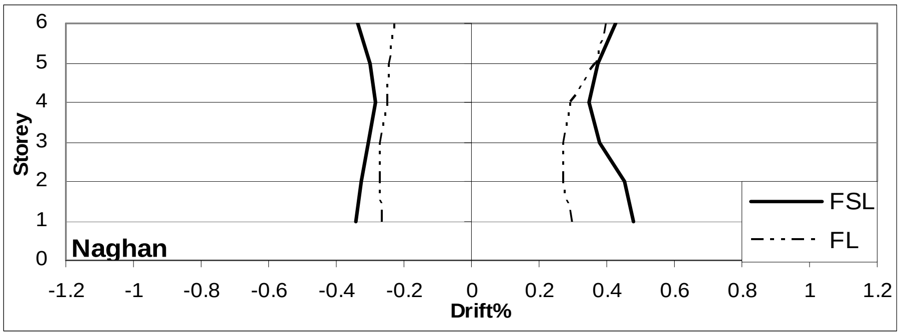


Fig. 12(a) Drift at each story for the 6-story building

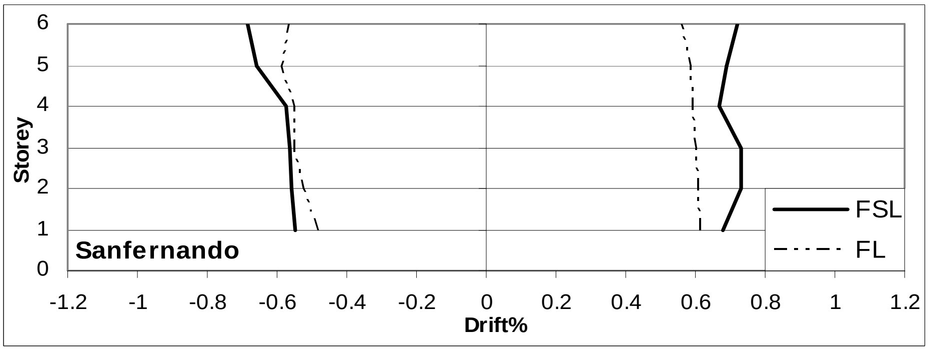


Fig. 12(b) Drift at each story for the 6-story building

Table3 Comparison of results for different earthquakes

		Ratio: $\frac{\textit{Max. relative parameter with anchorage slip}}{\textit{Max. relative parameter without anchorage slip}}$	
Earthquake	The kind of the building / Studied parameter	3-stories	6-stories
NAGHAN	Drift	109%	164%
	Inter Story Drift	115%	161%
TABAS	Drift	107%	115%
	Inter Story Drift	116%	126%
SANFERNANDO	Drift	163%	127%
	Inter Story Drift	169%	138%
EL-CENTRO	Drift	143%	153%
	Inter Story Drift	157%	160%

According to the results shown in Table 3, the difference between the drifts in the two cases of with and without slippage varies from 9 to 64% with an average of 35%. For the relative drifts; however, these values change from 15% to 69% with an average of 42%.

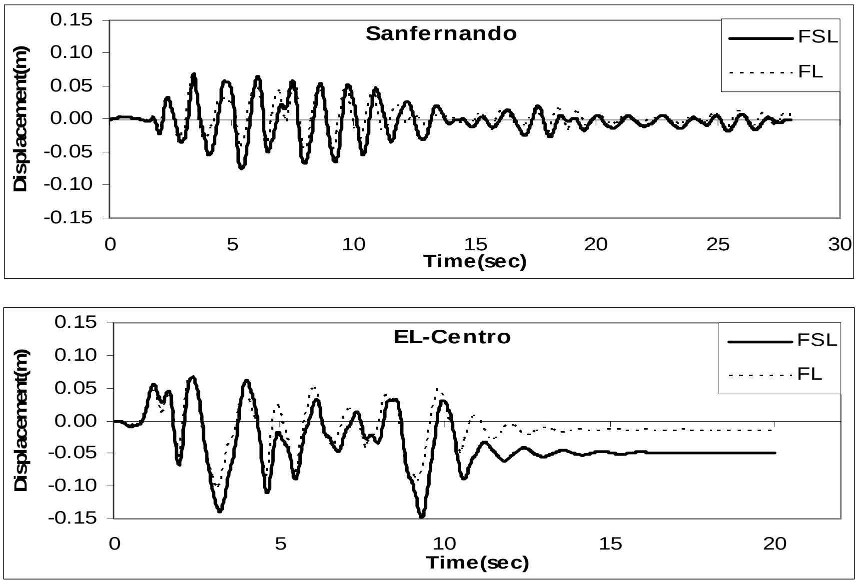


Fig. 13 Time history of roof displacement for different earthquakes in the 3-story building

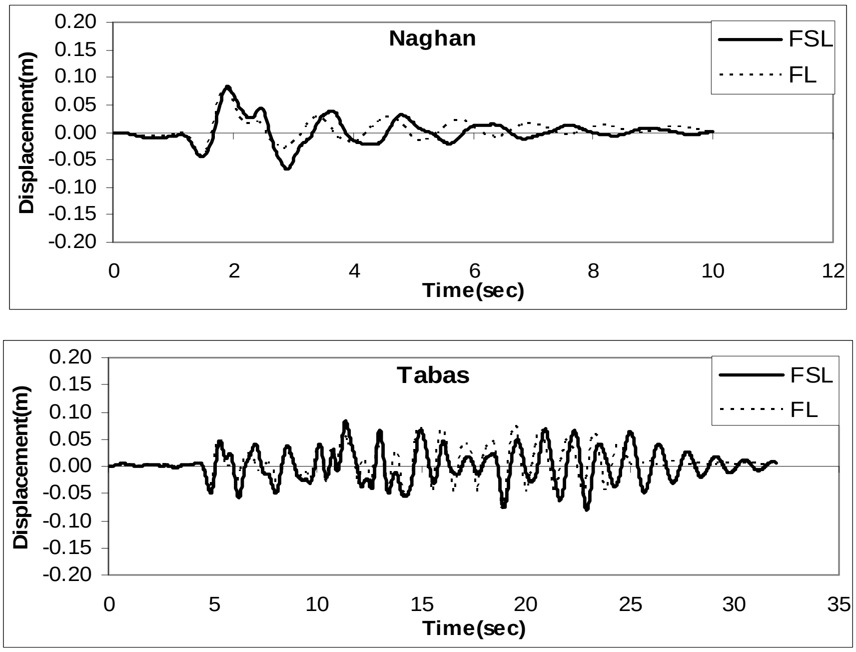


Fig. 14 Time history of roof displacement for the 6-story building

7. Time-history response of the roof displacement

The roof displacement time history is shown in Figs. 13 and 14 for the 3- and 6- story buildings subjected to different earthquake excitations with and without considering the bond slip effects.

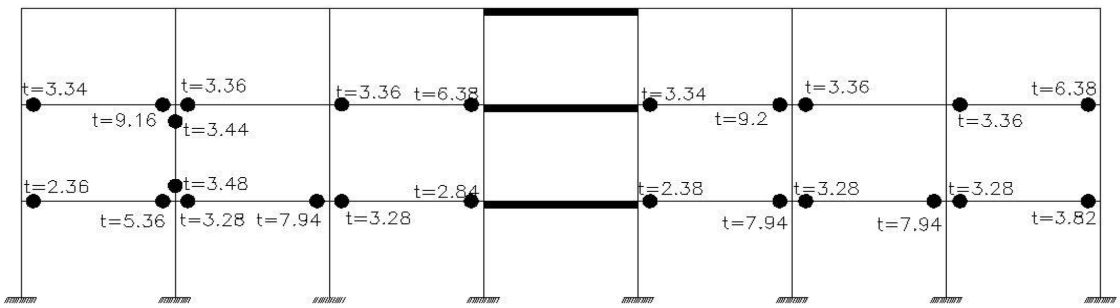
While the results diverge significantly for different earthquakes, the following conclusions were observed.

1. In all structures, when bond slip was considered in the analysis, the maximum roof displacement increased irrespective of the earthquake excitation used in the analysis.

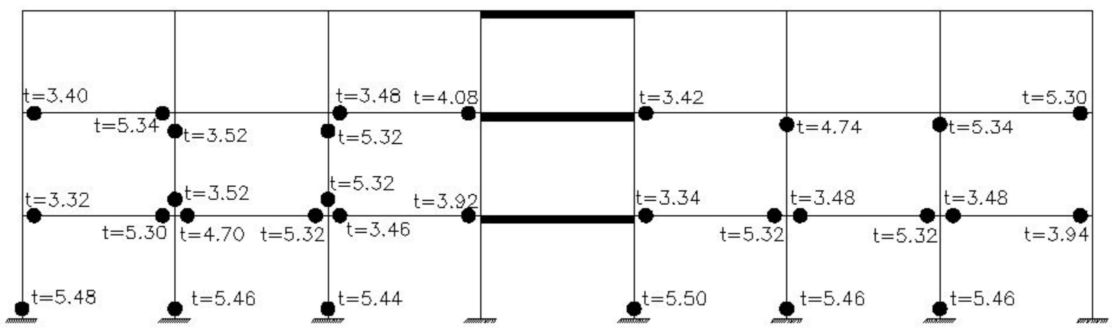
2. A phase change is seen between the cases with and without bond slip for the same earthquake so that maximum displacements have occurred at different times, in particular for the Naghan record.

8. Plastic hinging patterns

The study of plastic joint formation indicated that the incorporation of bond slip in the nonlinear analysis affected the time and sequence of joint formation in the members significantly. It should be mentioned that it normally does not affect the number and location of plastic hinges, albeit depending on the earthquake excitation applied. For example in cases using El-Centro and Sanfernando records resulted in a higher number of plastic hinge formation than Tabas and Naghan earthquakes. Some examples of plastic hinge patterns in different earthquakes are shown in Figs. 15 and 16.

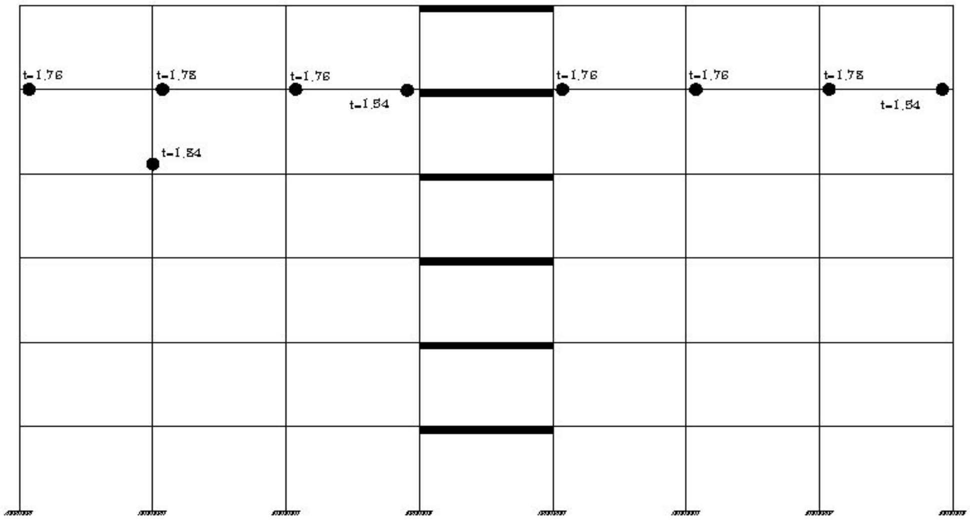


(a) Without anchorage slip

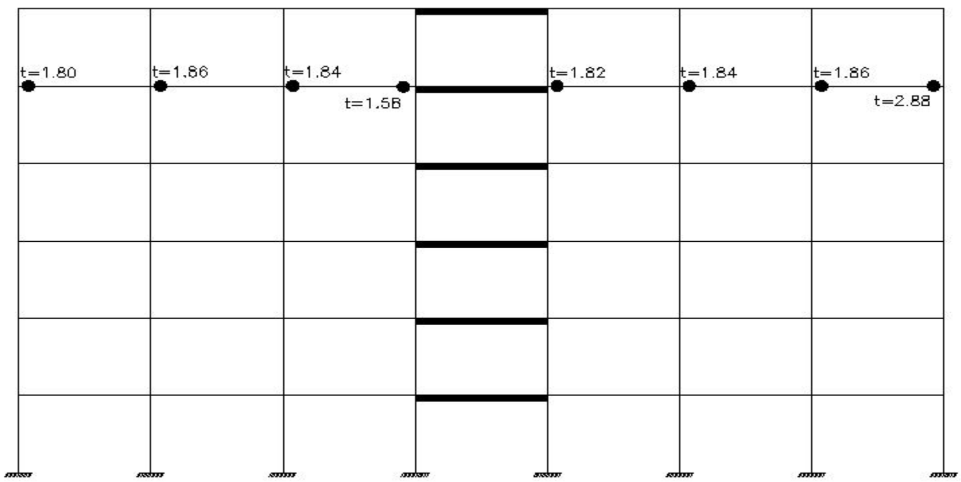


(b) With anchorage slip

Fig. 15 Plastic hinge patterns for the 3-story building subjected to the Sanfernando earthquake



(a) Without anchorage slip



(b) With anchorage slip

Fig. 16 Plastic hinge pattern of the 6-story building subjected to Naghan earthquake

9. Non linear pushover analysis: discussion of results

As second stage of this research, the two aforementioned buildings were studied using a push-over static analysis method by increasing the lateral load from zero to the value corresponding to the target drift defined by FEMA (2000). The results are shown in Figs. 17 - 20 for the two cases with and without considering the bond slip effects. As observed in the figures, structural deformations have increased significantly when bond slip deformations were considered.

The base-shear versus roof displacement is shown in Fig. 21. As it is seen, for the same base shear, higher displacements were produced for cases with bond slip.

Table 4 The results of carried comparisons in non- linear static analysis

		Ratio: $\dfrac{\textit{Max. relative parameter with anchorage slip}}{\textit{Max. relative parameter without anchorage slip}}$	
Analysis Type	Studied parameter	3 Stories	6 Stories
Nonlinear static	Drift	131%	120%
	Inter-story Drift	132%	122%

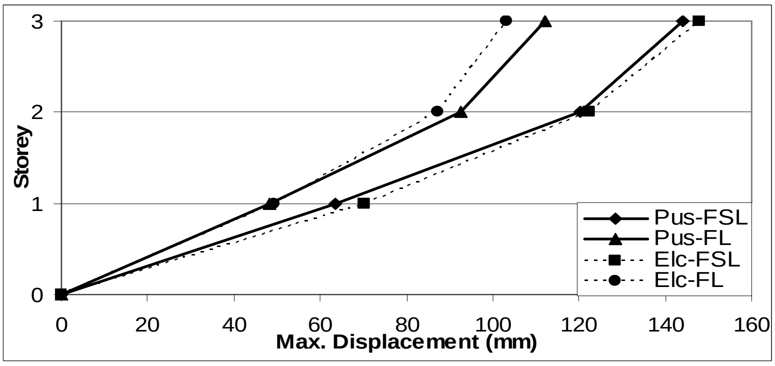


Fig. 17 Max. displacement in each story in the 3-story building

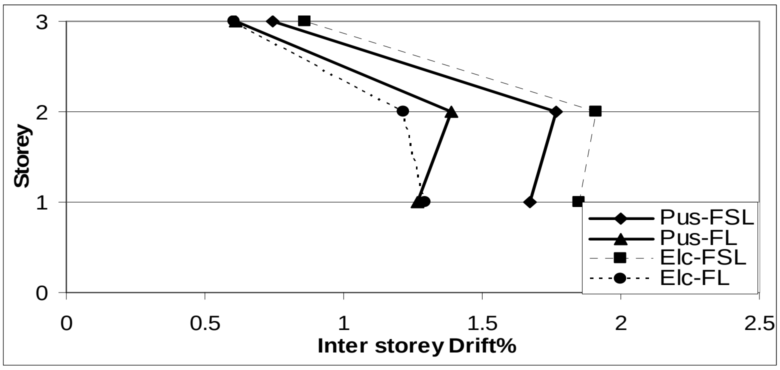


Fig. 18 Ratio of max. relative displacement of each story to the story height in the 3-story building

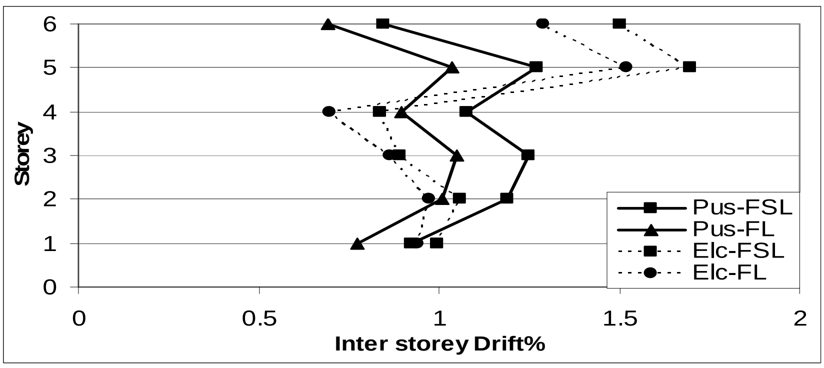


Fig. 19 Ratio of max. relative displacement of each story to the story height in the 6-story building

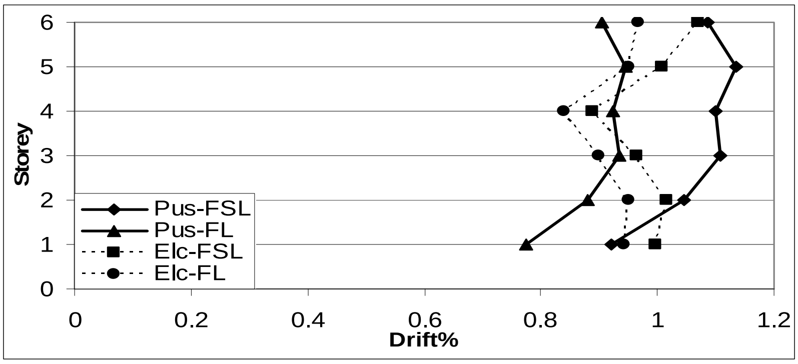


Fig. 20 Ratio of max displacement of each story to its height in the 6-story building

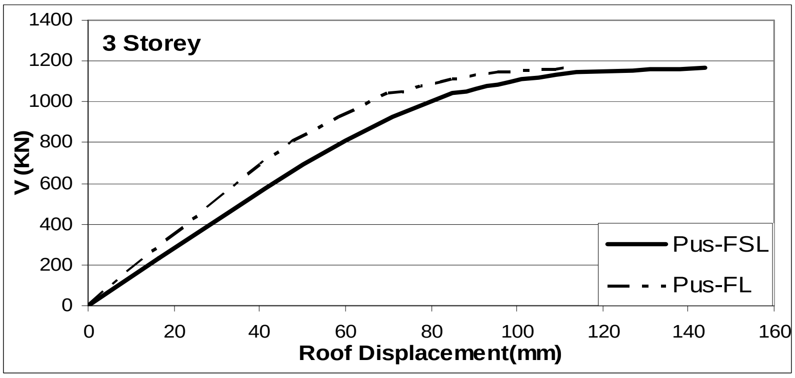


Fig. 21(a) Diagram of base shear in terms of roof displacement in the 3-story building

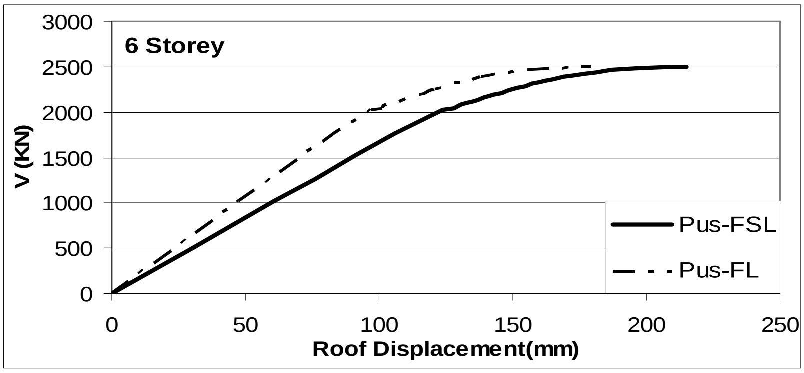


Fig. 21(b) Diagram of base shear in terms of roof displacement in the 6-story building

The results of the push-over analyses and time history analyses were closer to each other in the 3-story building than the 6-story building. This was to some extent predictable as the taller the building is, the higher is going to be the deviation of the actual inertia loading on the system from the simple load patterns used irrespective of the modal shapes of the structures in the pushover analysis. This matter was already observed and reported by other researchers (Shooshatri 1998).

10. Conclusions

In this study, the effect of the bond slip on the behavior of RC structures was studied when they are subjected to seismic loads.

1. A significant difference observed in the displacements, relative displacements and drifts between the two cases with and without considering the bond slip. For the structures analyzed, these differences were calculated to be around 35% for inter-story drift and 42% for relative displacement with variations that depend on the earthquake excitation used in the analyses and the height of the building.
2. The maximum displacements in the analyses with bond slip occurred at different times comparing to when no bond slip was allowed.
3. The location and number of plastic hinges change when the bond slip is considered also it varied with respect to its sequence and timing.
4. The graph of the base shear versus roof displacement showed that for the same base shear, higher displacements were observed when bond slip was considered.
5. The numerical results suggest that the loading capacity is not reduced by degradation of the bond-slip.
6. The results indicated that the bond demand along the joints is critical and that the rebar slippage inside the joint and in the footings results in large fixed-end rotations at the beam–joint interface.

References

Alsiwat, J.M., Ozcebe, G. and Saatcioglu, M. (1992), "Hysteretic behavior of anchorage slip in R/C members", *Journal of Structural Engineering, ASCE* ,**118**(9), 2439-2458.

Alsiwat, J.M. and Saatcioglu, M. (1992), "Reinforcement anchorage slip under monotonic loading", *Journal of Structural Engineering, ASCE ,* **118**(9), 2421-2438.

Ayoub, A. and Filippou, F.C. (1999), "Mixed formulation of bond-slip problems under cyclic loads", *J. Struct. Eng*., **125**(6), 661-671.

Drain-RC, (2006), "Inelastic analysis of R/C structures software", *Inter-Tech Engineering Service*, 1559 Alta Vista, Ottawa, Canada.

Dominguez, N., Fernandez, M.A. and Ibrahimbegovic, A. (2010). "Enhanced solid element for modeling of reinforced concrete structures with bond-slip", *Comput. Concrete*, **7**(4), 347-364.

Fallah, M.M. (2003), "Nonlinear dynamic analysis of concrete frames with considering anchorage slip", M.S. Thesis, Fredowsi University of Mashhad, Mashhad, Iran. (in Persian)

Federal Emergency Management Agency (2000), "Prestandard and Commentary for Seismic Rehabilitation of Buildings", FEMA-356, Washington, D.C.

Filippou, F.C., Popov, E.P. and Bertero, V.V. (1983), "Effect of bond deterioration on hysteretic behavior of reinforce concrete joints", *Earthquake Engineering Research Center*, University of California, Berkeley, Report No. EERC 83/19.

Iranian Code of Practice for Design of Concrete Buildings (ABA), 3rd Edition (2001), *Building and Hosing Research Center BHRC*, Tehran, Iran. (in Persian)

Iranian Code of Practice for Loading of Buildings, Standard No.519 (Revised Edition) (2001), *Building and Hosing Research Center BHRC*, Tehran, Iran. (in Persian)

Iranian Code of Practice for Seismic Resistant Design of Buildings (1999), Standard No. 2800, 2nd Edition, Building and Hosing Research Center BHRC-PN S 253. (in Persian)

Kwak, H.G. and Kim, J.K. (2006), "Implementation of bond-slip effect in analyses of RC frames under

cyclic loads using layered section method", *Journal of Engineering Structures*, **28**, 1715-1727.
Limkatanyu, S. and Spacone, E. (2002) "Reinforced concrete frame element with bond interfaces; Part I: Displacement-based,force-based, and mixed formulations", *Journal of Structural Engineering*, ASCE, **128**, 346-355.
Liu, Y. (2007), "A new method for steel-concrete composite beams considering interface slip", *Struct. Eng*., **23**(5), 35-40
Morita, S. and Kaku, T. (1984) "Slippage of reinforcement in beam-column joint of reinforced concrete frames", *Proceeding of the 8th World Conference on Earthquake Engineering,* San Francisco*,* **6**, 477-484.
Oh, B.H. and Kim, S.H. (2007), "Realistic models for local bond stress-slip of reinforced concrete under repeated loading", *J. Struct. Eng*., **133**(2), 216-224.
Otani, S. and Sozen, M. (1972), "Behavior of multistory reinforced concrete frames during earthquake", Structural Research Series No. 392*,* University of Illinois, Urbana, Illinois.
Ozcebe, G. and Saatcioglu, M. (1989), "Hysteretic shear model for reinforced concrete members", *Journal of Structural Engineering*, ASCE, **115**(1), 132-148.
Sezen, H. and Setzler, E.J. (2008), "Reinforcement slip in reinforced concrete columns", *ACI Structural Journal*, **105**(3), 280-289.
Shang, F., An, X., Kawai, S. and Mishima, T. (2010), "Open-slip coupled model for simulating three-dimensional bond behavior of reinforcing bars in concrete", *Computers and Concrete*, **7**(5), 403-419.
Shi, Y.C. and Li, Z.X. (2009), "Bond slip modelling and its effect on numerical analysis of blast-induced responses of RC columns", *Structural Engineering and Mechanics*, **32**(2), 251-267.
Shooshtari, A. (1998), "Seismic drift demands of reinforced concrete buildings", The Ph.D. Thesis, University of Ottawa in Civil Engineering, Canada.
Takeda, T., Sozen, M.A. and Nilson, N.N. (1970), "Reinforced concrete response to simulated earthquakes", *J. Struct. Div.,* ASCE, **96**(4), 2557-2573.
Wang, X. and Liu, X. (2009), "Predicting the flexural capacity of RC beam with partially unbonded steel reinforcement", *Computers and Concrete*, **6**(3), 235-252.
Yalcin, C. and Saatcioglu, M. (2000), "Inelastic analysis of reinforced concrete columns", *Computers and Structures*, **77**, 539-555.