Modeling of longitudinal passing bars within the joint panel in poor anchorage condition

Franco Braga, Rosario Gigliotti
DiSG - Dipartimento di Ingegneria Strutturale e Geotecnica, "Sapienza" Università di Roma, Via Eudossiana 18, 00185 Roma.

Michelangelo Laterza, Michele D'Amato, Suman Narsingh Rajbhandari
DiCEM - Dipartimento delle Culture Europee e del Mediterraneo: Architettura, Ambiente, Patrimoni Culturali, Via Lazazzera, 75100 Matera.

Keywords: bond-slip, interior joint, joint panel, moment-curvature relationships, nonlinear analyses.

ABSTRACT
In old structures reinforced with smooth bars and poorly detailed (because of insufficient anchorage length, poor confinement within critical regions) slippages of longitudinal bars become significant and govern the global response invalidating the classical full bond condition. Due to poor bond-strength capacity within the joint panel, passing-through longitudinal bars may result in tension throughout the panel joint and anchor within the opposite convergent element.
In this work the influence of anchorage loss of passing bars within joint panel is being investigated. Nonlinear analyses are being performed on an internal beam-column joint reproducing a connection of a concrete frame designed only for vertical loads and reinforced with smooth bars. The numerical investigations are compared with the experimental results of the joint that has been subjected to two consecutive tests: with and without FRP wraps applied at the columns zones near the panel joint. The sequence of the experimental tests has the aim of simulating a repairing procedure on a RC existing structure when a moderate seismic event occurs and a local strengthening at columns critical regions is designed.

1 INTRODUCTION

Much of the building constructed in the Mediterranean countries before 1970's are non-seismically designed reinforced concrete (RC) frame structures as these structures were designed and detailed considering only the gravity load. The inadequacy of these existing structures has been highlighted by heavy damage or total collapse caused by the recent destructive earthquakes (Friuli 1976, Kochaeli 1999, Irpinia 1980, L'Aquila 2009).

One of the principal reasons of damaging of these structures during earthquake is contributed by the damaging of beam-column joints. Their behaviour is a crucial aspect in the seismic design of RC structures because in these zones high stresses transfer among the converging elements for satisfying equilibrium conditions. In the case of seismic resistant frames passing steel bars are subjected near the panel joint to tensile forces at one side and compressive forces at the opposite one requiring the bar anchorage within the joint. In the case of poor bond conditions, very common in old RC structures, the anchorage within the panel joint cannot be fully satisfied and, consequently, a reduction of the flexural capacity of the converging elements may occur (Hakuto et al., 1997; Fabbrocino et al., 2004).

It is therefore very important to develop models capable of considering properly the behaviour of beam-column joints for accurately predicting the global behaviour of the structure. This work is addressed to investigate the influence of the loss of anchorage of passing longitudinal bars on the response of RC sub-assemblages. Numerical investigations and comparisons with the experimental results are performed on an internal beam-column joint designed only for vertical loads and reinforced with smooth bars. The specimen has been subjected to two consecutive tests by applying a constant vertical load and a lateral displacement history. In the first test the lateral displacement history has been applied up to a certain level of damage considered acceptable. Then, the columns of the joint have been strengthened with FRP wraps and the specimen has been re-tested up to the failure. The tests sequence aims to reproduce a repairing procedure applied to columns of RC structure when a moderate seismic event occurs, and the failure behaviour is characterized by strong columns-weak beams mechanism. The FRP wraps provide an improvement of confinement action within columns critical regions without any modification of the failure mechanism. In the nonlinear analyses bond-slips of longitudinal bars are taken into account with the modified steel stress-strain relationship developed by Braga et al. (2012).

2 STATE OF ART

In traditional analyses of the RC moment resisting frames, beam-column intersection zones are idealized as rigid connections. This assumption can be unrealistic because if the passing bars cannot transfer all the forces by bond, significant slippages take place within the joint panel. This implies an increasing of the lateral deformation of the structure. Sezen et al. (2002), Hakuto et al. (2000), for example, have concluded that contribution of slips on lateral deformation of column is significant and they give rise to a loss of energy dissipation capacity.

There have been published several works addressed to simulate RC beam-column behaviour. The simplest approach for modelling joint response within the context of a nonlinear frame analysis is to introduce a spring at the intersection of the beam and column line elements (Otani 1974, Anderson 1977, El-Metawally et al. 1988, Alath et al. 1995, Biddah et al. 1999).

The macro-element models represent an alternative approach for modelling the joint behaviour. Many researchers have proposed models connecting beams and columns to a finite-volume joint macro-element (Youssef et al. 2001, Calvi et al. 2002, Lowes et al. 2003, Altoontash et al. 2003, Shin et al. 2004). These models comprise a shear-panel component and rotational springs or zero-length springs to represent bar slip and shear interface.

3 ANALYTICAL FORMULATION

Under the earthquake loading, the beams and columns adjoining a joint are subjected to moments in same (clockwise or anticlockwise) direction as shown in Figure 1. Under these moments, each passing bar is at one face in tension and at the opposite one in compression. The axial force state has to be balanced by bond stresses that develop between concrete and steel in the joint region-(Hakuto et al. 1999).

The problem of the reinforcing longitudinal bar of diameter D passing through an interior beam-column joint of length L_N can be analysed by considering forces equilibrium as shown in the Figure 2 and Figure 4 with the application of monotonically increasing slip at the right end of the section (tension side).

The proposed analytical modelling of the bar is based on the same assumptions of the simplified model proposed by Braga et al. (2012):

1. Bond-slip field along the reinforcing bar *u(x)* is linear along the bar;
2. Bond stress-slip relationship is perfectly elastic-plastic.

Following the first assumption, bond slip at any rebar section *u(x)* within the joint panel is given by the equation:

$$u(x) = u_C + \frac{(u_T - u_C)}{L_N} x \qquad (1)$$

Instead, as regards the second assumption, the bond stress-slip relationship can be expressed as:

$$\tau(x) = \tau_d \frac{u(x)}{u_1} \rightarrow u(x) \le u_1 \qquad (2a)$$

$$\tau(x) = \tau_d \rightarrow u(x) > u_1 \qquad (2b)$$

where u_C and u_T are the axial displacements of the bar at the compression and the tension faces, respectively; τ_d is the ultimate bond strength and u_1 the axial displacement of the bar at ultimate bond strength.

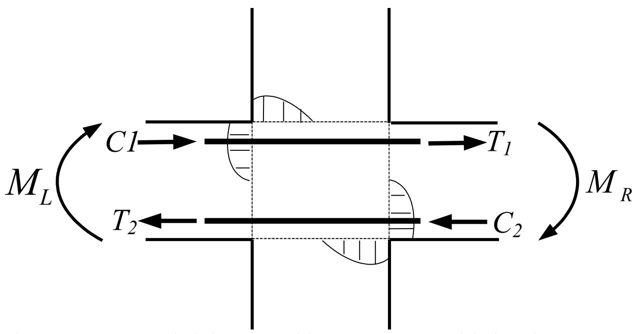

Figure 1. Bar axial forces of beams at panel joint faces.

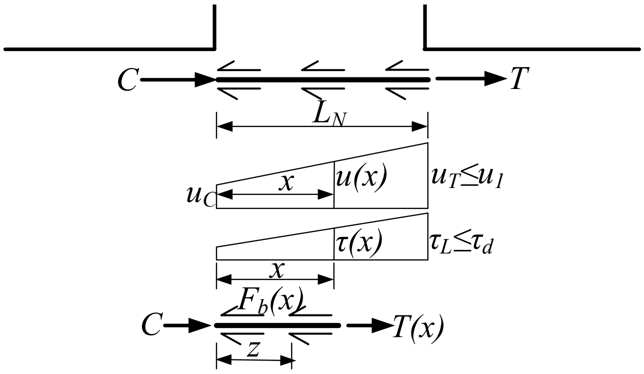

Figure 2. Equilibrium of the passing bar for $u_T \leq u_1$.

Let us consider the slip at the right end is less than or equal to the slip at ultimate bond strength as shown in the Figure 2.

For $u_T \leq u_1$, the equation of equilibrium of the bar portion x can be written as:

$$C + T(x) = F_b(x) \tag{3}$$

where C is the compression at left face, and T(x) is the tension at abscissa x.

The bond force $F_b(x)$ can be calculated as:

$$F_b(x) = \int_0^x \tau(z)\pi D dz \tag{4}$$

where $\tau(z)$ is bond strength; D diameter of passing steel bar; x is the part of bar under consideration.

By substituting Eq. (4) and (2) into Eq. (3) and by referring to the overall length L_N, the equation of the ratio C/T is obtained (Eq. 5).

$$\frac{C}{T} = \frac{\tau_d \pi D L_N}{u_1 T}\left[\frac{(u_C + u_T)}{2}\right] - 1 \tag{5}$$

Let $T = \alpha A_b f_y$ then the Eq. (5) can be re-written as:

$$\frac{C}{T} = \frac{\tau_d \pi D L_N}{\alpha A_b f_y}\left[\frac{(u_C + u_T)}{2u_1}\right] - 1 \tag{6}$$

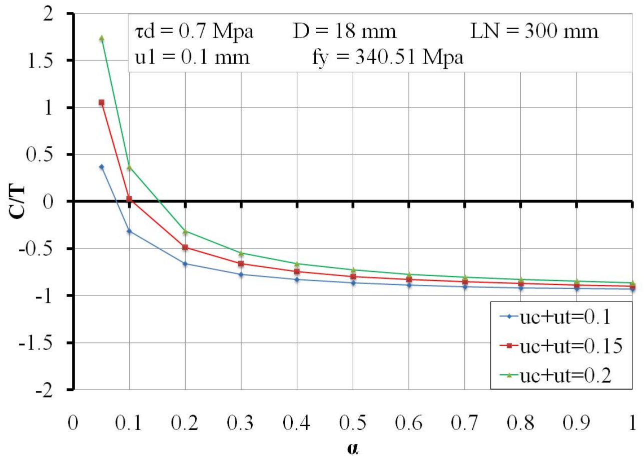

Figure 3. C/T ratio when $u_T \leq u_1$.

The Eq. (6) gives the ratio C/T as function of the slippages at the two joint faces and of the tensile force at one end of the bar. In Figure 3 the Eq. (6) is reported by considering different values of slips at two faces of the panel joint and referring to a 18 mm bar diameter. When C/T is negative passing bar at both faces is in tension.

Figure 4 shows the equilibrium of the passing bar when the ultimate bond strength is partially reached within the panel joint, that is when $u_C < u_1 < u_T$.

When $x \leq L_N - L'$, the variation of bond stress is linear as given by Eq. 2(a). Hence the formulation for this zone is similar to the above formulation.

When $x > L_N - L'$, the bond-strength includes both the contribution of the linear variation portion and that of the rectangular portion of the constant branch as shown in Figure 4. In this case:

$$F_b(x) = \tau_d \pi D\left[x - (L_N - L')\right] + \int_0^{L_N - L'} \tau(z)\pi D dz \tag{7}$$

Starting from Eq. (7) and substituting the Eq. (2) the ratio C/T may be derived as below.

$$\frac{C}{T} = \frac{\tau_d \pi D L'}{T} + \frac{\tau_d \pi D}{u_1 T}\left[\frac{(u_C + u_1)(L_N - L')}{2}\right] - 1 \tag{8}$$

By assuming:

$$T = \alpha\left(A_b f_y\right) \quad 0 \leq \alpha \leq 1 \tag{9}$$

$$L' = \beta L_N \quad 0 \leq \beta \leq 1 \tag{10}$$

The Eq. (8) becomes:

$$\frac{C}{T} = \frac{\tau_d \pi D L_N}{A_b f_y}\left[\frac{\beta}{\alpha} + \frac{(u_C + u_1)}{2u_1}\frac{(1-\beta)}{\alpha}\right] - 1 \tag{11}$$

where u_C can range between 0 and 0.1.

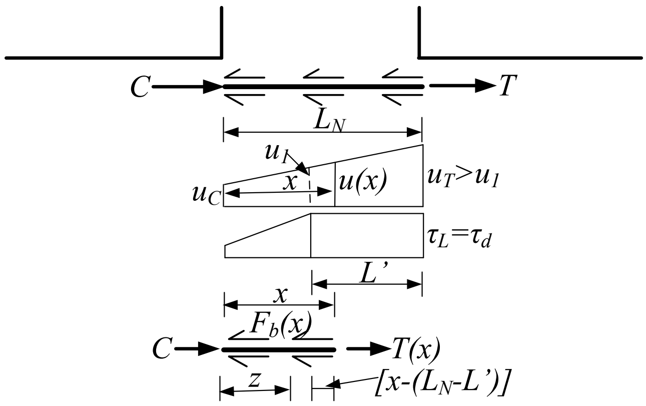

Figure 4. Equilibrium of the passing bar for $u_C < u_1 < u_T$.

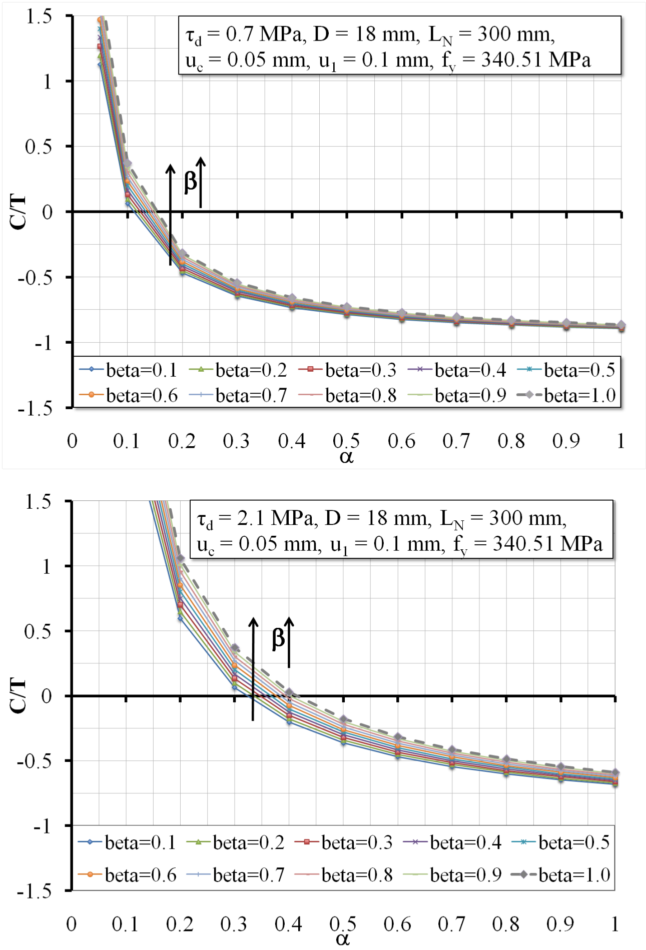

Figure 5. C/T of passing bar for u_C = 0.05 a) for τ_d=0.7 MPa, and b) τ_d=2.1 MPa.

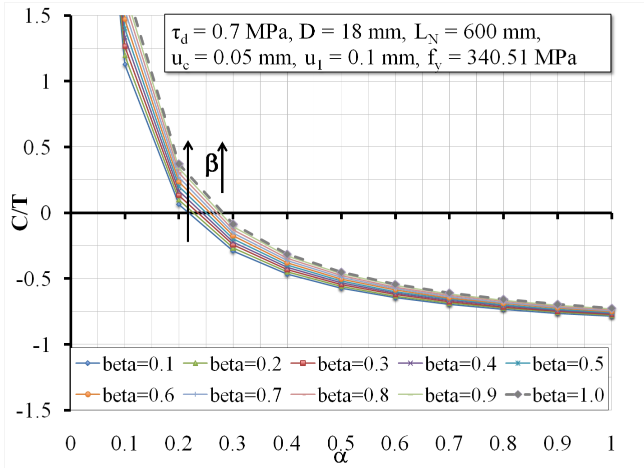

Figure 6. C/T ratio of passing bar for L_N=600 mm.

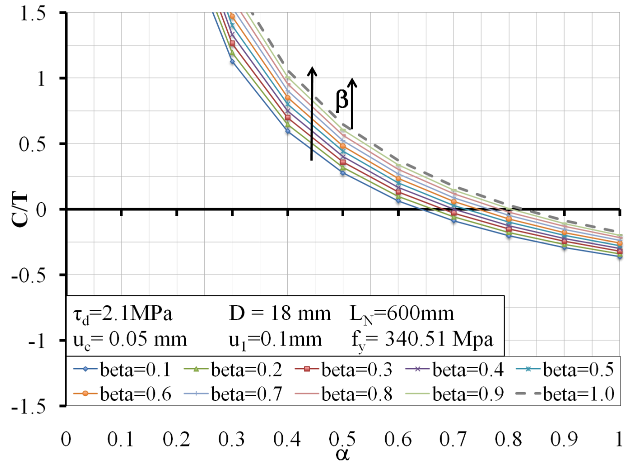

Figure 7. C/T of passing bar for τ_d=2.1 and L_N=600 mm.

Eq. 11 can be plotted for different values of τ_d as shown in Figure 5. In this figure the ratio *C/T* is represented by varying the ratio α, and for different values of β (the length of the embedded bar where the ultimate bond strength is reached). As it is clear to understand, the ratio *C/T* decreases if the tensile ratio of the steel bar increases. Moreover, after a certain tensile force in the bar, the compressive force *C* reduces and ultimately changes in tension (negative values of *C/T* ratios).

In the Figure 5 the stress reversal takes place at very low stress ratio of α due mainly to the poor bond conditions and to the short anchorage of the bar. The stress reversal at the compressed side can be delayed: by increasing the bond strength (Figure 5b); by increasing the anchorage length of joint (Figure 6); or increasing both (Figure 7).

In all figures has been reported the ratio *C/T* when the ultimate bond strength within the panel joint is attained (dashed line). In this case the equilibrium equation (Eq. 11) becomes:

$$C + T = \pi D L_N \tau_d \tag{12}$$

4 FLEXURAL STREGTH IN THE CASE OF LOSS OF ANCHORAGE

As discussed in the previous paragraphs with the increasing of moments, the compression force within the steel rebar can start in decreasing due to the insufficient anchorage length within the joint panel. Thus, the compression force at the opposite side can turn into tension although the bar is placed into the element concrete compression zone (Figure 8). This implies larger compressive area of concrete and, therefore, a changing of lever arm and a reduction of the flexural strength of the element converging at the joint (Shiohara 2001, Fabbrocino et al. 2004).

In order to investigate the influence of reduction of longitudinal bars compressive strength on the flexural capacity moment-curvature analyses have been performed on a column section with 500mm by 500mm dimensions having 8 20mm diameter bars. For this purpose three stress-strain relationships with a different value of strength in compression have been considered: full strength in compression, 50% of strength in compression, and no strength in compression. Figure 9 compares the resulting interaction diagrams, which clearly show the influence of the reduction of strength in compression when loss of anchorage arises within the panel joint.

5 NUMERICAL INVESTIGATIONS AND COMPARISONS

The influence of the loss of anchorage on the response of RC structures has been investigated by referring to an interior beam-column joint designed only for vertical loads and reinforced with smooth bars.

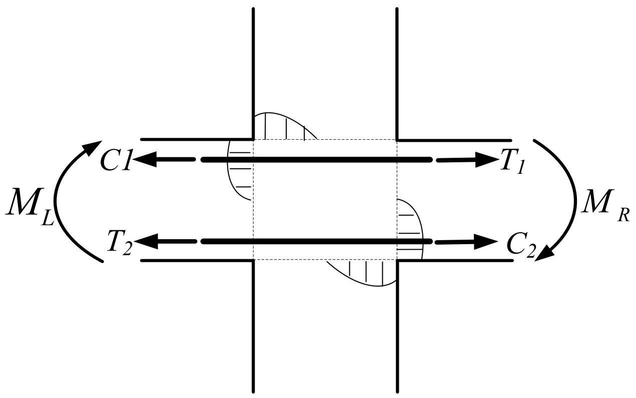

Figure 8. Forces along the bar when tensile force occurs at both joint faces.

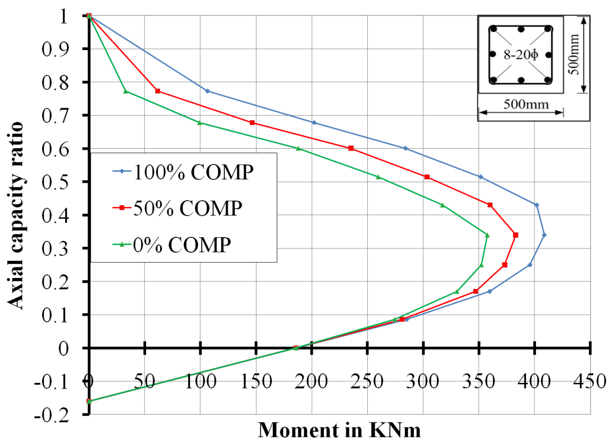

Figure 9. Interaction diagram with different percentage of compression branch in steel constitutive model.

The specimen considered, has been subjected to a vertical axial load and to a lateral displacement history showing a flexural hinging of the columns near the panel joint. After this test, the same specimen has been repaired by strengthening the columns regions with FRP wraps and re-tested in the same way until the failure. This repairing procedure, easily applied if moderate damaging due to weak column-strong beam behaviour arises, is addressed to improve the confinement within critical regions of columns without any modification of the failure mechanism.

The specimen (joint "C11") considered in this work has been tested within an experimental campaign regarding beam-column joints reproducing connections of an old 2D RC structure (Braga et al. 2009).

In the Figure 10 is compared the experimental relationship between the lateral force of the actuator and the imposed horizontal displacement at the top of the upper column of the joint C11 with and without FRP wraps.

5.1 *Numerical simulations and comparison with experimental results*

To perform numerical simulation of the interior beam column joint C11, the joint has been modelled in the OpenSees software (Mazzoni et al. 2006). The beams and columns are modelled with BeamWithHinges elements which consider plasticity to be concentrated over specified hinge lengths at element ends. For analyses, this plastic hinge has been assumed to be equal to h/3.

The joint panel zone is modelled by rigid elements and elastic trusses are used for representing the test apparatus.

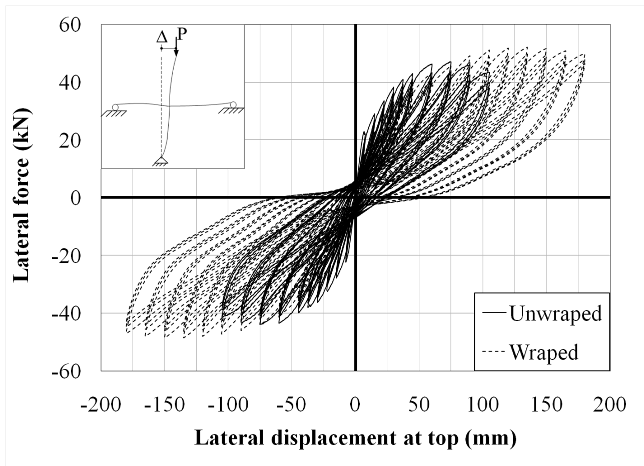

Figure 10. Experimental force-displacement relationship for C11 joint w/o wrapped columns.

In the non linear analyses the simplified model developed by Braga et al. (2012) is used. This model provides a relationship for longitudinal steel bars incorporating slippage of the reinforcement with respect to the surrounding concrete. The model is particularly appropriate for non linear analyses of old RC buildings with poor anchorage conditions due, for example, to the application of smooth bars and negligible confining action in the critical zones. The model describes the behaviour of straight or hooked bars and is developed starting from the basic assumption that bond-slip along the bar can be described with a linear field. Applications of the model can be found in D'Amato et al. (2012a).

Three different steel stress-strain relationships are adopted in the numerical simulations: full bond law, bond-slip law with 100% strength in compression and bond-slip law with 0% strength in compression. For confining effects due to steel hoops and FRP wraps are accounted with the BGL model (D'Amato et al. 2012b). Stress-strain relationships of concrete are depicted in Figure 12.

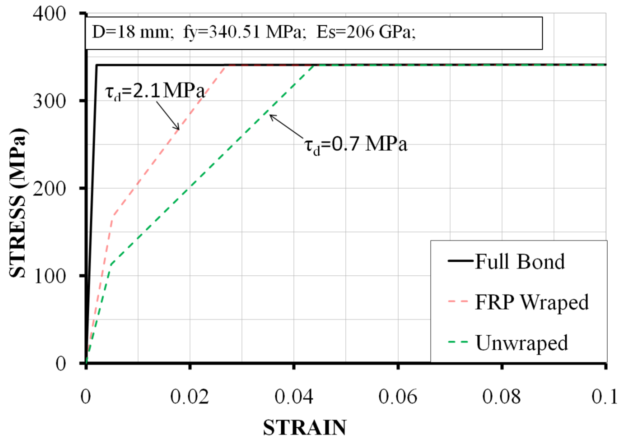

Figure 11. Stress-strain relationships for steel.

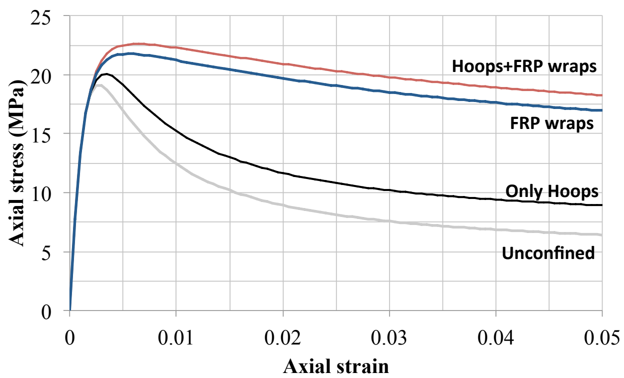

Figure 12. Stress-strain relationships for concrete.

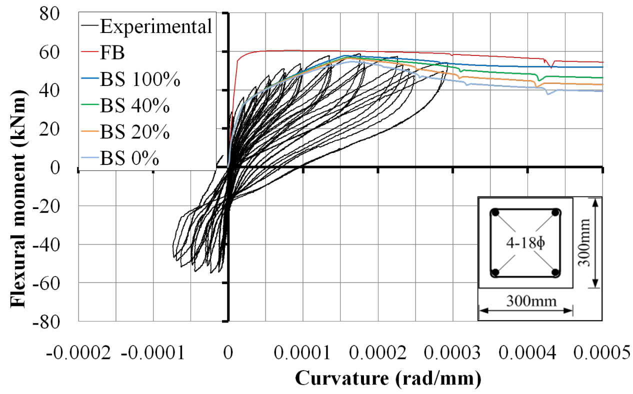

Figure 13. Experimental and analytical moment-curvature diagram for upper column section.

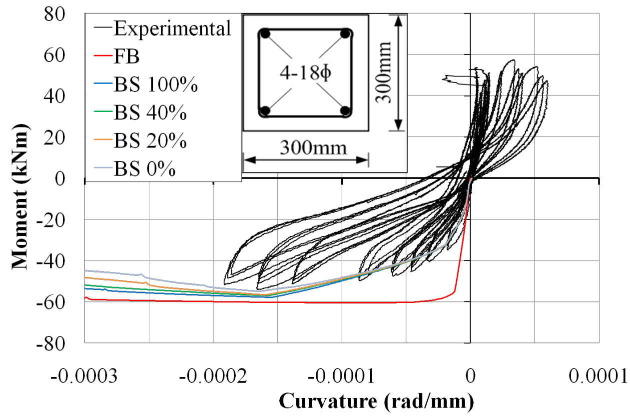

Figure 14. Experimental and analytical moment-curvature diagram for lower-column section.

In Figure 13 and Figure 14 are reported moment-curvature comparisons of the upper and lower column section with the experimentally derived ones (Braga et al. 2009).

The numerical analyses have been conducted on a fiber section considering different strengths in compression of longitudinal bars. The steel stress-strain relationships incorporate the bond-slips by applying the simplified model proposed by Braga et al. (2012). The considered stress-strain relationships of longitudinal steel are: full bond, bond-slip law with 100%, 40%, 20% and 0% strength in compression. The moment-curvature analyses are obtained by considering the axial load ratio of 16%. The comparisons show that the bond-slip significantly reduces the stiffness of the section and, in this case, it delays the yielding of longitudinal bars. Bond-slip also drastically reduces the flexural strength when high axial load acts on the RC section (Figure 15).

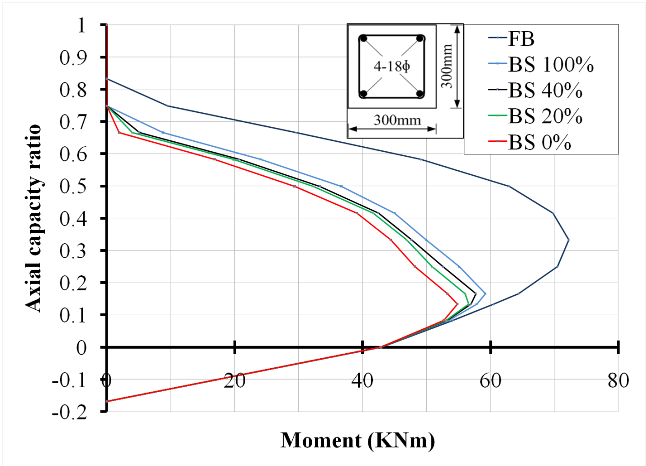

Figure 15. Interaction diagram with and without bond slip.

5.2 *Lateral force-displacement relationships*

The lateral force-displacement relationships at the top of C11 are reported in Figure 16 through Figure 18. They show the comparisons between the experimental and analytical response by considering for longitudinal passing bars two different stress-strain relationships: bond-slip law with 100% and 0% strength in compression. The latter relationship of longitudinal bars simulates the loss of anchorage within the panel joint of the passing bars. The comparison among the responses envelopes is reported in Figure 22.

It can be clearly noticed that full bond assumption overestimates the strength, stiffness and energy dissipation of the joint. By considering slippages and no strength in compression of columns passing bars a better agreement with the experimental results is being achieved (Figure 19 through Figure 21).

The specimen C11 has been tested up to a reparable drift ratio. At the end of the test the joint has been repaired by improving the confinement within columns regions near the panel joint where inelastic excursions occurred.

At this scope three layers of uniaxial CFRP wraps with a width of 1.5 times the section depth have been applied.

After the repairing procedure the specimen has been re-tested until the failure. The local strengthening has not changed the failure mechanism with respect to the first test. In fact, it has been still observed a hinging of columns near the joint panel.

Figure 23 through Figure 25 are reported comparisons of the numerical simulations of the wrapped joint with the experimental response. It is important to note that the confinement improves as well the bond strength within the critical regions of the passing bars. For this reason, it has been assumed in these analyses that the bond strength of passing bars is $3\tau_d$, where τ_d is the value of the bond strength assumed in the previous analyses without FRP wraps (stress-strain relationship assigned to columns passing bars are reported in Figure 11).

The cycle-by-cycle comparisons (Figure 26 through Figure 28) show the influence of the loss of anchorage for larger values of drift ratios, when slippages of passing bars are considerable.

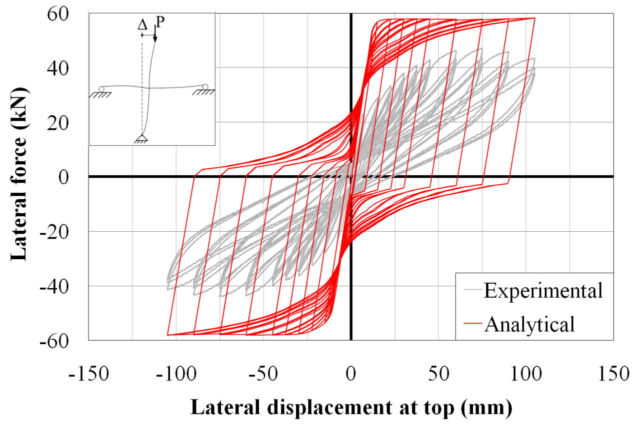

Figure 16. Lateral force-displacement response of the joint without wraps with full bond steel law.

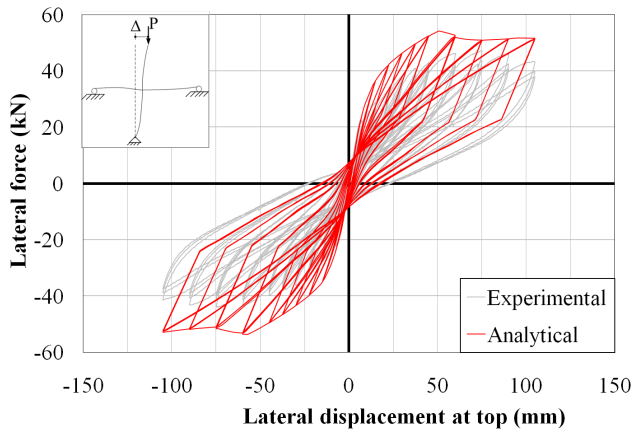

Figure 17. Lateral force-displacement response of the joint bond-slip with 100% strength in compression.

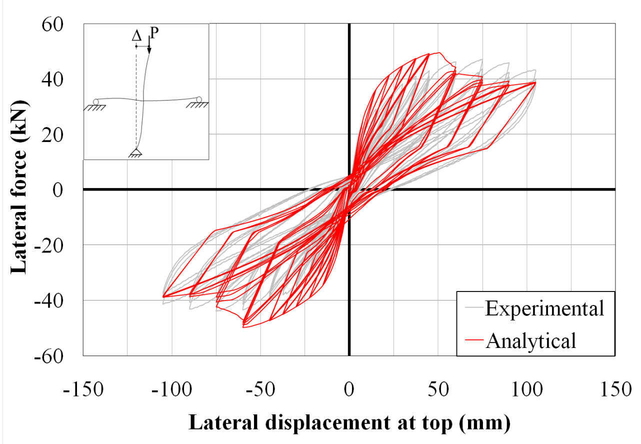

Figure 18. Lateral force-displacement curve for bond-slip with 0% strength in compression.

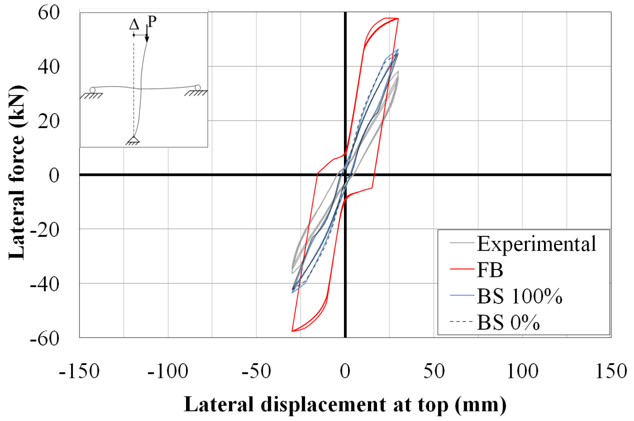

Figure 19. Lateral force-displacement curve for 30mm cycles.

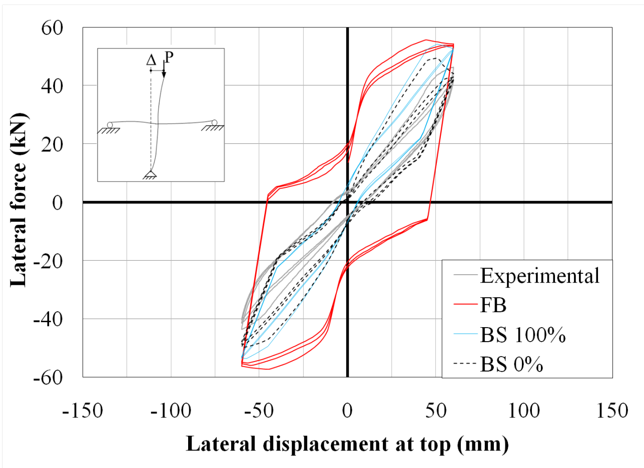

Figure 20. Lateral force-displacement curve for 60mm cycles.

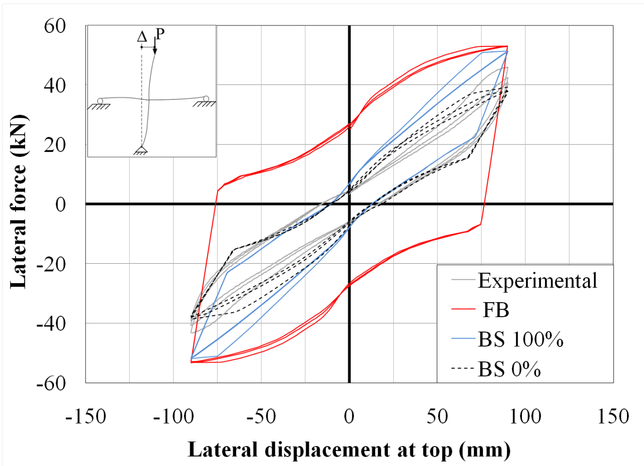

Figure 21. Lateral force-displacement curve for 90mm cycles.

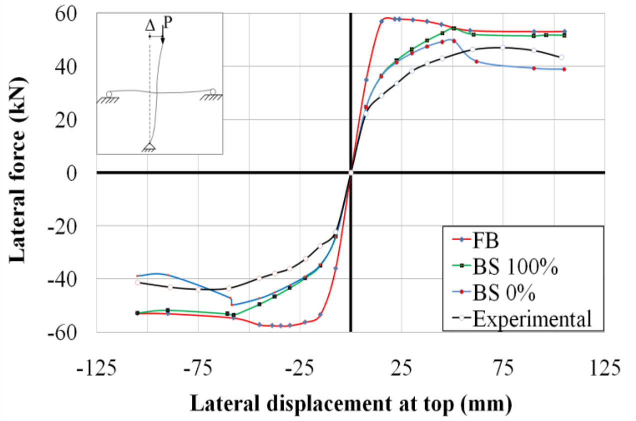

Figure 22. Analytical and experimental lateral force-displacement envelope curves.

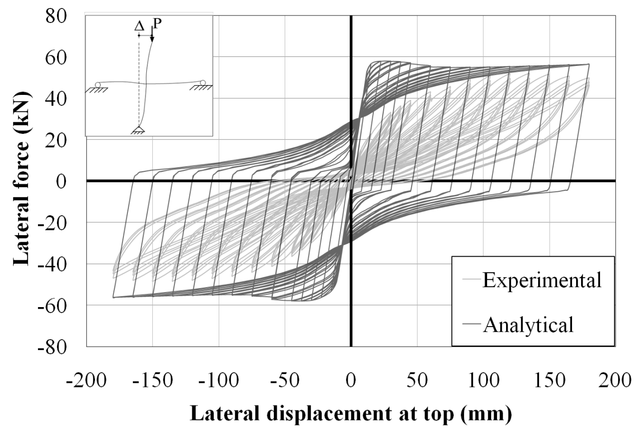

Figure 23. Lateral force-displacement curve for joint with FRP wrap with full bond.

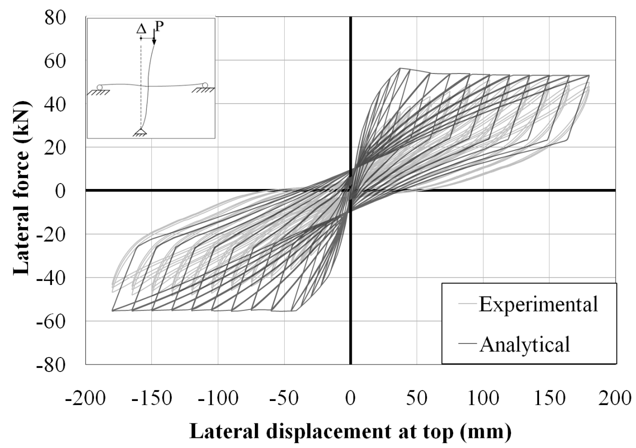

Figure 24. Lateral force-displacement curve for joint with FRP wrap with bond-slip with 100% compression.

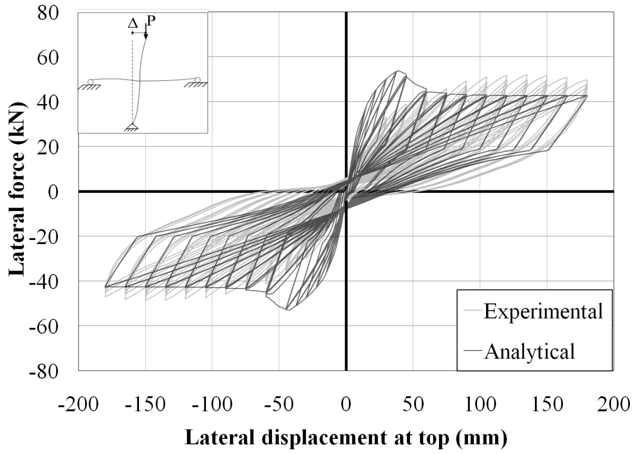

Figure 25. Lateral force-displacement curve for joint with FRP wrap with bond-slip with 0% compression.

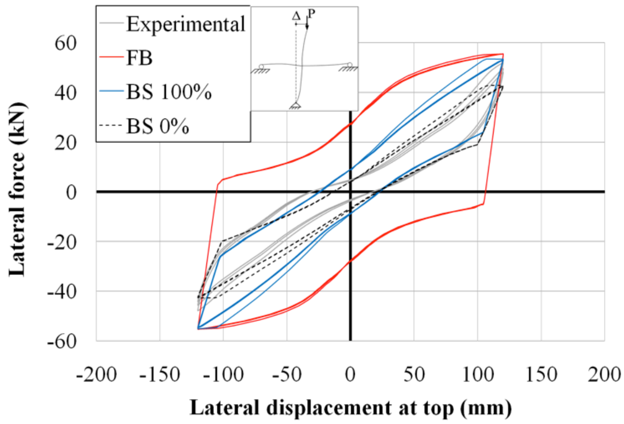

Figure 26. Lateral force-displacement curve for 120mm cycles in joint with FRP wrap.

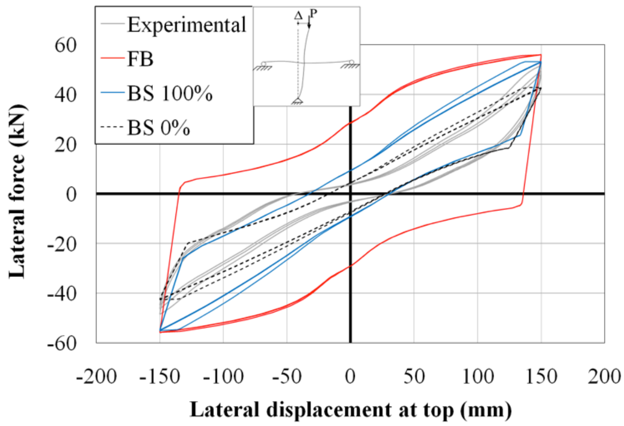

Figure 27. Lateral force-displacement curve for 150mm cycles in joint with FRP wrap.

6 CONCLUSIONS

In this paper the influence of the insufficient anchorage length of longitudinal bars passing through the joint panel has been investigated.

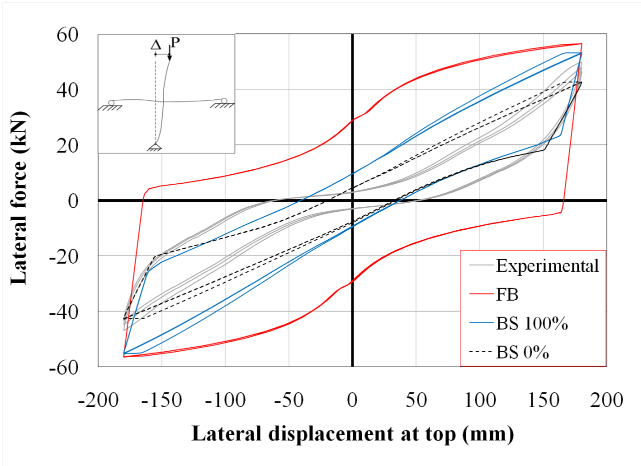

Figure 28. Lateral force-displacement curve for 180 mm cycles in joint with FRP wrap.

The numerical investigations carried out confirm the importance of this aspect particularly important in poorly detailed structures reinforced with smooth bars. Because of the loss of anchorage within the joint panel high slippages of rebars take place within the critical regions dominating the lateral response.

The preliminary study shown in this paper represents a first step in proposing analytical models accounting for the loss of anchorage of passing bars. As shown, it reduces the flexural strength especially when the axial load is high. This means that this phenomenon can modify the failure local mechanisms and facilitate flexural yielding of columns rather than beams. Therefore, an analytical model reproducing this aspect will allow us to reproduce more realistically the response of RC structure.

Starting from the analysed preliminary considerations, different analytical models may be developed with a different level of refinement. For example, a plastic hinge model based on the description of the phenomenon at the section level or a finite element of the joint panel may be developed. The latter should be capable to properly describe the loss of anchorage describing the interaction between the steel fibers of two opposite sections modelling the same passing bar.

Finally, the degradation of the bond strength within the joint panel represents another aspect to be investigated especially in the case of deformed longitudinal bars. It should be modelled defining a damaging and an unloading/reloading law for non-linear cyclic analyses.

REFERENCES

Alath, S., Kunnath, S., 1995. Modelling Inelastic Shear deformation in RC Beam-Column Joints. *Proceeding of*

10th Conference on Engineering Mechanics, University of Colorado at Boulder, Boulder, pp. 822-825.

Altoontash, A., Deierlein, G.D., 2003. A versatile model for beam-column joints. *ASCE Structures Congress,* Seattle.

Anderson, J.C., Townsend, W.H., 1977. Models for RC Frames with Degrading Stiffness. *Journal of the Structural Division, ASCE*. 103 (ST12): 1433-1449.

Biddah, A., Ghobarah, A., 1999. Modelling of shear deformation and bond slip in reinforced concrete joints. *Structural Engineering and Mechanics,* 7(4): 413-432.

Braga, F., Gigliotti, R., Laterza, M., 2009. Existing RC structures with smooth bars: experimental behaviour of beam-column joints subject to cyclic lateral loads. *The Open Construction & Building Technology Journal,* 3(1), 52-67.

Braga, F., Gigliotti, R., Laterza, M., D'Amato, M., Kunnath, S., 2012. Modified Steel Bar Model Incorporating Bond-Slip for Seismic Assessment of Concrete Structures. *Journal of Structural Engineering, ASCE,* 138(11), pp.1342-1350.

Calvi, G.M., Magenes, G., Pampanin, S., 2002. Relevance of beam-column joint damage and collapse in RC frame assessment. *Journal of Earthquake Engineering,* Vol. 5, Special Issue 1 (2002) 75-100.

D'Amato, M., Braga, F., Gigliotti, R., Laterza, M., Kunnath S., 2012a. A numerical general purpose confinement model for non-linear analysis of R/C members. *Computers and Structures Journal, Elsevier,* Vol. 102-103, pp. 64-75.

D'Amato, M., Braga, F., Gigliotti, R., Kunnath, S., Laterza, M., 2012b. Validation of a Modified Steel Bar Model Incorporating Bond-Slip for Seismic Assessment of Concrete Structures. *Journal of Structural Engineering, ASCE,* 138(11), pp.1351-1360.

El-Metwally, S.E., Chen, W.F., 1988. Moment-rotaion modelling of reinforced concrete beam-column connections. *ACI Structural Journal,* 85, 4.

Fabbrocino, G., Verderame, G.M., Manfredi, G., and Cosenza, E., 2004. Structural models of critical regions in old-type R.C. frames with smooth rebars. *Engineering Structures,* 26(14), 2137-2148.

Hakuto, S., Park, R., Tanaka, H., 1999. Effect of deterioration on bond of beam bars passing through interior beam-column joints on flexural strength and ductility. *ACI Structural Journal,* 96(5), pp.858-864.

Hakuto, S., Park, R., Tanaka, H., 2000. Seismic Load Tests on Interior and Exterior BeamColumn Joints with Substandard Reinforcing Details. *ACI Structural Journal,* 97(1), pp.11-25.

Lowes, L. N., Altoontash, A., 2003. Modeling reinforced concrete beam-column joints subjected to cyclic loading. *Journal of Structural Engineering, ASCE*.

Mazzoni, S., McKenna, F., Scott, M.H., Fenves, G.L., Jeremic, B., 2006. Opensees Examples Primer. University of California.

Mazzoni, S., McKenna, F., Scott, M.H., Fenves G.L., 2007. OpenSees Command Language Manual, *University of California.*

Sezen, H., Whittaker, A.S., Elwood K.J., Mosalam K.M., 2003 Performance of reinforced concrete buildings during the 17 August 1999. Kocaeli, Turkey earthquake, and seismic design and construction practise in Turkey. *Engineering Structures*, 25:103-14.

Otani S., 1974. SHAKE-A Computer Program for Inelastic Response of R/C frames to Earthquakes, *University of Illinois* at URBANA.

Sezen, H., Moehle, J.P., Bond-slip behaviour of reinforced concrete members. (This work was funded by the *Pacific Earthquake Engineering Research (PEER) center* through the *Earthquake Engineering Research Centers* Program of the *National Science Foundation*).

Shin, M., LaFave, J.M., 2004. Modeling of cyclic joint shear deformation contributions in RC beam-column connections to overall frame behaviour. *Structural Engineering and Mechanics,* Vol. 18, No. 5 (2004) 645-669.

Shiohara, H., 2001. New Model for Shear Failure of RC Interior Beam-Column Connections. *Journal of Structural Engineering,* Vol. 127, No. 2.

Youssef, M., Ghobara, A., 2001 Modelling of RC Beam-Column joints and Structural Walls. *Journal of Earthquake Engineering,* (1), 93-111.