FRAGILITY CURVES FOR REINFORCED CONCRETE FRAMES WITH LAP-SPLICED COLUMNS

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Abstract   Placing column lap-splice in the locations of possible nonlinear deformation may adversely affect the structures response to strong ground motions. Localization of damage in splice zone may change the structural response and prevent the load redistribution and development of a uniform pattern of nonlinear excursions among the various members. Validated by existing laboratory experiments, this study presents a model that could be used to evaluate the behavior of lap-spliced columns. The proposed model is able to include the effect of the longitudinal bars arrangement; bars yield stress, and the amount and spacing of transverse bars. Comparison with existing experimental tests, show a good correlation between the model and experimental results. Finally, to obtain an estimation of the importance of the bar slip in lap splice on the structures response, fragility curves for life safety and collapse limit states are developed for an ordinary moment resisting frame of a one bay-ne story structure. Incremental dynamic analysis is used to derive the fragility curves. These fragility curves show that the bar slip have significant impact on the probability of exceeding collapse limit state, while its impact on the life safety limit state is not so significant.

Keywords   Lap-Splice, Slip, Incremental Dynamic Analysis, Fragility Curve

چکیده   قرارگرفتن وصلهٔ ستون‌ها در محل تغییرشکل‌های محتمل غیرخطی، ممکن است عملکرد سازه را حین وقوع زلزله‌های بزرگ تحت‌تأثیر قرار دهد. موضعی‌شدن آسیب در محل وصله ممکن است به تغییر رفتار سازه منجر شود و از بازتوزیع بار و رسیدن به الگوی یکنواخت رفتار غیرخطی در کل سازه جلوگیری کند. این مقاله درصدد عرضهٔ مدلی برای بررسی عددی رفتار ستون‌های وصله‌شده است. نتایج به‌دست آمده با نتایج کارهای آزمایشگاهی مقایسه شده است. مدل پیشنهادی اثر آرایش آرماتورهای طولی، تنش تسلیم آرماتورها، و تعداد و فاصلهٔ آرماتورهای عرضی را در نظر می‌گیرد. مقایسه با نتایج آزمایشگاهی نشان‌دهندهٔ همبستگی خوبی بین نتایج تحلیلی و تجربی است. در پایان برای بررسی اهمیت لغزش وصله در رفتار سازه‌های بتنی، منحنی شکست برای حدود ایمنی جانی و فروریزش برای یک قاب خمشی بتنی یک‌طبقهٔ یک‌دهنه ارائه می‌شود. برای محاسبهٔ این منحنی‌های شکست از نتایج آنالیز دینامیکی فزاینده استفاده شده است. این مطالعه نشان می‌دهد که لغزش وصله تأثیر به‌سزایی روی حد فروریزش دارد؛ درحالی‌که اثر آن روی حد ایمنی جانی اندک است.

1. INTRODUCTION

One of the common deficiencies in the existing buildings designed before 1970s is the short length of splice and the lack of good confinement in the lap splice length. Based on the typical design procedure at the time, the lap splice was designed to accommodate only compression load transfer. On the other hand, large spacing of transverse reinforcements in these structures is not able to confine the concrete to provide the required force transfer between spliced bars. The use of 90 degrees hooks in transverse reinforcement (suspect to opening after spalling of cover concrete) decreases even more the efficiency of confinement provided by transverse bars. This typical detail results in non-ductile response of the column and is one of the main causes of failure in previous earthquakes [1,2]. The common practice of placing the columns lap splice just above the floor slab or foundation, where nonlinear deformations in columns may happen, worsens the situation.

Placing columns lap splice at column toe for buildings designed as ordinary or intermediate moment resisting frames is permitted in the current ACI code [3]. Therefore, in these structures to have an estimation of the importance of the bar slip in lap-spliced columns on the structural response, there is the need for models capable of simulating the effect of probable bar slip.

Most of the studies on lap-spliced bars are focused on the maximum load that can be developed in the spliced bars (e.g. [4,5]). Supported by these findings, it is usually assumed that the strength of the splice is composed of two parallel terms. The first term exhibits the contribution of unconfined concrete to the lap splice capacity, while the second term depicts the contribution of confinement provided by transverse reinforcement. However, there is an agreement in research communities that the second term contribution in the case of cyclic loading is negligible [6].

In studies that consider the effect of bar slip on the deformation, the focus was on the rigid body rotation due bar elongation and slip in the joints known as the strain penetration effect. Sezen, et al [7] using uniform bond stress distribution with different values in elastic and yielded regions, reached to an estimation of the amount of the column rigid body rotation due to strain penetration in the beam-column or column-foundation joints. Zhao, et al [8] developed a zero length element capable of modeling the effect of strain penetration.

In a leading study, Eligehausen, et al [9] based on extensive experimental studies, proposed a bond stress-slip model, which includes an ascending, yielding, softening and finally constant stress region. After initial focus of experimental researches on the maximum transferable load in the spliced bars, now there exist extensive experimental researches on the effect of the lap splice on the load deformation behavior of the reinforced concrete elements and subassemblies (e.g. [10-12]). However, there is still the lack of analytical works on the subject in the literature. Xiao, et al [12] following their experimental studies, proposed a model for the effect of confinement provided by composite materials on the bond stress-slip model for spliced bars under monotonic loading. Suggesting a softening bond

stress-slip model, they assumed that the maximum bond stress is a function of the amount of confining stress provided by composite materials, which seems reasonable for the case of confinement provided by the composite materials, while this is not the case for confinement provided by transverse reinforcement as mentioned earlier.

Ghobarah, et al [13], ignoring the increase in deformation caused by bar slip in lap splice, only modeled the effect of bar slip on the flexural strength of columns. They concluded that softening in the element response initiated by bar slip could lead to significant increase in the lateral displacement demand of the structure.

Han, et al [14] experimentally investigated the behavior of ordinary and intermediate concrete moment resisting frames (OMRF and IMRF), where main emphasis is placed on the effect of lap splice in the columns performance. They used 2/3 scale model specimens in their investigations, however due to known size effect in the bar slip (e.g. [15,16]), their results should be used with caution. They concluded that presence of bar slip has no appreciable effect on the hysteretic response of OMRF and IMRF exterior columns, while for interior columns, IMRF columns perform slightly better than OMRF ones.

Pincheira, et al [17] modeled the cyclic behavior of columns using concentrated nonlinear rotational springs at member end to model stiffness and strength degradation, and pinching effects. They used two rotational springs in series to model nonlinear rotation due to flexural deformations and slip occurring in lap splice length. They used Harajli, et al [18] model to obtain monotonic behavior of the spring for bar slip. Then by trial and error method and comparison with experimental results, they obtained the value of the parameters required for cyclic response (stiffness and strength degradation, and pinching parameters). Their model working in macro level is numerically efficient; however, main problem with it lies in obtaining the parameters required for definition of the rotational spring properties.

Paspuleti [19] tried to simulate the performance of Holiday Inn hotel located at San Fernando Valley, approximately 4.5 miles from the epicenter of the 1994 Northridge earthquake. The building, which was an old reinforced concrete structure, suffered extensive damage in the Northridge

earthquake. To model the lap splice in his model, he used the splice model of FEMA 356 [20]. In the FEMA 356, the maximum stress in spliced bars is assumed to be proportional with available splice length. Also, it is assumed that splice can maintain the maximum stress at larger deformations if the spacing of the lateral reinforcements be smaller than one-third of section depth, otherwise the maximum developed stress should be degraded to 20 percentages of maximum stress in ductility demand equal to 2. He concludes that the FEMA 356 model does not capture the splice failures occurred in some of the ground story columns during the Northridge earthquake.

On the other hand, fragility curves become increasingly more popular in engineering communities as an appropriate tool for decision-making. Fragility curves provide information on the conditional probability of exceeding a given damage state for earthquakes of different intensities. There are large amount of literature on developing fragility curve for structure and for structural and nonstructural elements as well, however there is limited attempts to develop the fragility curves for non-ductile reinforced concrete structures. Mosalam, et al [21] used pushover analysis to derive the fragility curves for low-rise reinforced concrete frames designed only for gravity loads. Rossetto, et al [22] also developed fragility curve for reinforced concrete frames designed according to code in place in 1982 in Italy. They also used pushover analysis in their work. Celik [23] developed the fragility curves for non-ductile reinforced concrete frames susceptible to mid America ground motions, where he used results of dynamic analysis to develop the fragility curve however; he ignored the effect of bar slip in lap splice.

This study is going to evaluate the effect of the arrangement of longitudinal bars, and the amount and the spacing of transverse reinforcement on the performance of lap-spliced bars. For this purpose, first a model capable of accounting for bar slip in lap splice is set forward and then the results of the proposed method have been compared with existing experimental results. The paper concludes with development of the fragility curves for life safety and collapse limit states of a one bay-one story OMRF, where incremental dynamic analysis is employed to derive the fragility curves.

2. PROPOSED MODEL FOR STRESS-STRAIN OF LAP-SPLICED BARS

In this section, it is intended to develop a model for stress-strain behavior of the lap-spliced bars. To do this, first the stress-strain of the spliced bar in monotonic loading is established and then using experimental results the degrading and pinching effects of cyclic loading is taken into account (discussed in the next section). The proposed stress-strain curve takes into account the arrangement of longitudinal bars, the amount and spacing of transverse bars and their yield stresses. Figure 1 depicts the proposed stress-strain curve for lap-spliced bars. In this figure, $f_{s,max}$ is maximum stress developed in lap-spliced bars, f_r is residual stress also referred to as frictional stress, ε_s is strain corresponding to $f_{s,max}$ and ε_r denotes strain corresponding to reaching to the residual stress zone.

To obtain the value of stresses $f_{s,max}$ and f_r the method of Priestley, et al [24] is used. Providing that the maximum stress in splice is proportional to the tensile strength of concrete, they assumed a failure surface around each spliced bar with height equal to lap-splice length and perimeter equal to p. Bars maximum developable force can be obtained by multiplying the area of this failure surface by tensile strength of concrete. Supposing that the slip resistance is provided with truss mechanism of 45 degrees between spliced bars or between bar and surrounding concrete, this tensile force will be equal to bond force in spliced bar (Figure 2). Therefore, the maximum force developed in lap-spliced bar will be

$$T_b = A_b f_s = F_t p l_s \qquad (1)$$

Where T_b is the force developed in lap-spliced bar, A_b is bar cross section, f_s is bar stress, F_t is tensile strength of concrete ($0.33\sqrt{f_c'}$ MPa, where f_c' denotes stress corresponding to the concrete compressive strength), p is perimeter of cylindrical block, and l_s is lap-splice length. Perimeter of the above-mentioned failure surface for rectangular columns becomes

$$p = \frac{s}{2} + 2\left(d_b + c\right) \leq 2\sqrt{2}\left(c + d_b\right) \qquad (2)$$

Where s is average distance between spliced bars.
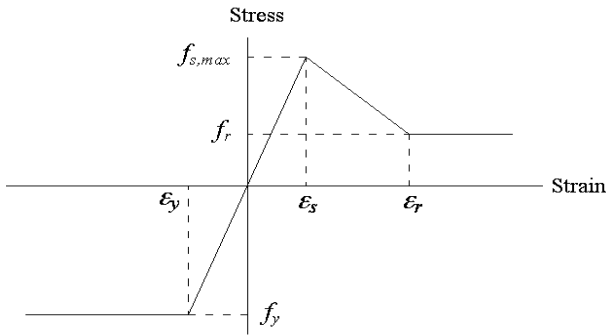
Figure 1. General form of proposed stress-strain curve for lap-spliced bar.
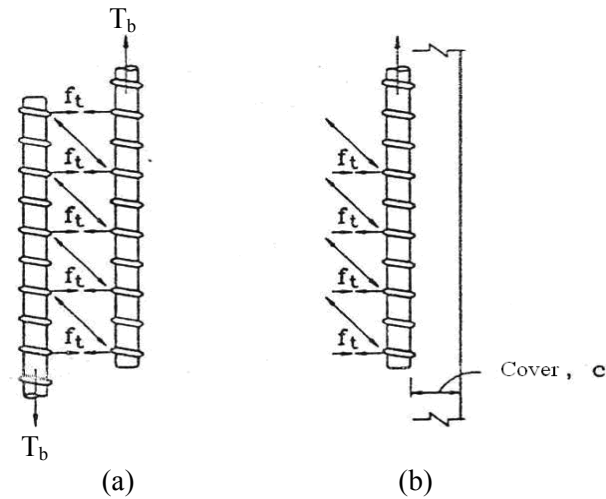
Figure 2. Tension stresses induced by force transfer in lap-splices (Priestley, et al [24]), (a) Between bars and (b) Between bar and core.

The upper limit in this equation applies for widely spaced spliced bars. This derivation ignores any increase in maximum developable stress due to confinement provided by transverse reinforcements. This is in agreement with the behavior observed under cyclic loading as discussed in the introduction (MacKay, et al [6]).

After reaching to the point of maximum stress, softening initiates that continue until reaching to constant residual stress zone, which corresponds to frictional stress developed in failure surface. Using the shear-friction concept it is assumed that the transverse reinforcements crossing the crack plane provide the necessary friction to transfer bond

stresses. With friction factor μ on failure surface, the frictional stress on crack plane can develop the following stress in longitudinal bars of rectangular columns

$$n_l n_t \mu A_h f_h = n A_b f_r \qquad (3)$$

Here n_l denotes number of transverse reinforcement legs perpendicular to crack plane, n_t is number of transverse reinforcements in lap-splice length, A_h is the area of crack surface, f_h is yield strength of transverse reinforcement with a maximum of $0.015 E_s$ where E_s is bar's modulus of elasticity. In this equation n is number of spliced longitudinal bars developed by frictional stress in the crack plane. In the following calculations, frictional factor, μ, is taken equal to 1.4.

In this study, it is assumed that displacement due to slip occurs in a length equal to l_{ss}, which may be different from splice length. This is a fictitious length, which may be smaller or larger than the actual splice length (as will be discussed in the next paragraph). At peak stress, total strain in spliced bar is the summation of the bars elastic strain, ε_{se}, and the strain due to bar slip, ε_{ss}. So, at peak stress total strain, ε_{st} becomes

$$\varepsilon_{st} = \varepsilon_{se} + \varepsilon_{ss}$$
$$\varepsilon_{se} = f_{s,max} / E_s \qquad (4)$$
$$\varepsilon_{ss} = u / l_{ss}$$

Where u is bar slip. In this study, it is assumed that slip corresponding to maximum stress is 1 mm and slip corresponding to reaching to the frictional stress (roughly equal to bar lug spacing) is 10 mm.

Softening behavior observed in the response of lap-spliced bars, is an indication of presence of size effect in the experimental results. In fact experimental results of Sener, et al [15] and Ichinose, et al [16] substantiate this. Nevertheless, the proposed model does not address size effect. Presence of size effect is an indication of the importance of comparing the results with experiments done with real dimensions not scaled one. On the other hand, due to softening, the analysis results will be mesh-dependent (Bazant, et al [25]). Extending the concept of crack band to the flexural members Bazant, et al [26] and Bazant, et al [27] suggest that to avoid mesh dependency in

numerical analysis the minimum mesh size should be taken equal to the section depth. Therefore, in this study the length of the element that models the bar slip (i.e. l_{ss}) is set equal to the section height.

3. FINITE ELEMENT MODEL OF COLUMNS WITH LAP-SPLICE

This study employs OPENSEES [28] for performing finite element analyses. This software has a perfect material and element library, and is capable of doing nonlinear pushover and response history analyses. To model the lap-spliced bars, we have used two fiber elements in each column in series. One of these elements (hereafter called splice element) models the deformations in splice due to flexure and bar slip, while the second one (hereafter called flexural element) models the flexural deformations in the rest of the column. For each of these elements, the element called nonlinear beam column in OPENSEES element library is used. This is a force-based element capable of satisfying equilibrium in the element length in the case of occurrence of softening in the member ends. For numerical integration Gauss-Lobatto scheme, with two Gauss points in splice element and three in flexural element is used. The section behavior in each Gauss point is obtained using fiber sections, where analysis ignores any shear deformation. Behavior of concrete was modeled with concrete01 material, while for reinforcements of flexural and splice elements, steel01 and hysteretic materials employed, respectively. The hysteric material is used to model the tri-linear behavior of spliced bar as depicted in Figure 1 and its formulation uses Park-Ang's damage model (Park, et al [29]) to simulate cyclic degradation. In this formulation the damage parameter is considered as a linear additive combination of maximum displacement and dissipated energy

$$s_d = (1 - \delta s)s_0$$

$$\delta s = a_1 \left(\frac{d_{max}}{d_f} \right) + a_2 \left(\frac{E_{hys}}{E_{mon}} \right) \qquad (5)$$

Here s denotes strength or stiffness of section, δs is

the amount of strength or stiffness degradation, d_{max} is maximum displacement, d_f is displacement corresponding to failure, E_{hys} is absorbed energy in cyclic loading, E_{mon} is absorbed energy in monotonic loading, and a_i factors are selected to have best agreement with experimental results. The hysteretic material also can consider the pinching effects. To obtain proper cyclic parameters, after performing numerous analyses and comparing the analytical results with experimental ones, best correlation is found for energy and displacement damage parameters equal to 0.02 and 0.0, respectively. Also to model the observed pinching in the experimental results the pinching parameters in the strain and stress axes is set equal to 0.8 and 0.3, respectively.

4. COMPARISON WITH EXPERIMENTAL RESULTS

Analytical results using proposed method is compared with experimental results of Melek, et al [10], and Aboutaha, et al [11]. The properties of the specimens used in this comparison are shown in Table 1. As can be inferred from this table, in comparison with new codes, the specimens have short lap-splice length in the range of 20-24 times longitudinal bar diameter. The performance of these specimens is an indication of the possible performance of the columns in buildings designed before adoption of seismic requirements in the codes. For all specimens, failures were due to loosing bond between concrete and reinforcements in splice length, and there are little cracking in the foundation. This fact shows that yield penetration due to reinforcement stressing in the foundation that lead to rigid body rotation at column-foundation juncture, is not significant. Therefore, in this study the effect of strain penetration is ignored. In the analyses, all of the columns were subjected to unidirectional cyclic loading using the same displacement history that was applied during the tests. It should be noted that movement and rotation of the foundation block or P-Δ effects were not included in the model as the reported lateral load and displacement histories of the columns was already corrected for these effects. The analyses on the specimens have been done using displacement-control.

The stress-strain curve parameters of the proposed model for each specimen are given in Table 2. As can be seen, in the specimens with short lap-splice length $(20d_b)$, longitudinal bars are not able to reach to the yield stress. Also note that due to inadequacy of the lateral reinforcement in the splice length after maximum stress the bars are unable to maintain the maximum stress in larger strains and frictional stresses are about 21 to 33 percent of the maximum stress developed in the spliced bars. Figure 3 indicates the comparison between analytical and experimental results, which

shows that there is good correlation. The proposed approach is able to model strength, softening slope and stiffness degradation with good accuracy.

To obtain a better comparison, in Figure 4 results of monotonic analyses by the proposed model are compared against some of the experimental results. The figure also shows the performance of the FEMA 356 model. As may be inferred from this figure the proposed model work well in estimating important parameters of the response, while those of FEMA model is very conservative as may be expected because it is a design code.

TABLE 1. Experimental Specimens used for Comparison in this Study.

| Researcher	Column	Column Dimension			Long. Bars			Trans. Bars		Concrete	Axial Load
		b, in	h, in	l, in	No.	l_s, in	f_y, Ksi	Spacing	f_{yh},Ksi	f_c', Ksi	
Aboutaha	FC4	36	18	108	16#8	24	63	#3@16	58	2.85	0
	FC14	27	18	108	12#8	24	63	#3@16	58	4.17	0
	FC15	18	18	108	8#8	24	63	#3@16	58	4.17	0
Melek	S10MI	18	18	72	8#8	20	74	#3@12	69	5.26	$0.1A_g f_c$'
	S20MI	18	18	72	8#8	20	74	#3@12	69	5.26	$0.2A_g f_c$'
	S30MI	18	18	72	8#8	20	74	#3@12	69	5.26	$0.2A_g f_c$'

Notes: b = width of column cross section; h = height of column cross section; l = clear height of column;
l_s = splice length; f_y = yield strength of reinforcing bar; f_c' = compressive strength of concrete;
A_g = gross area of column cross section; 1 in. = 25.4 mm; and 1 ksi = 6.89 MPa.

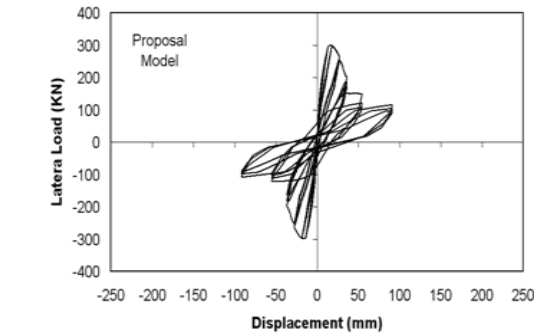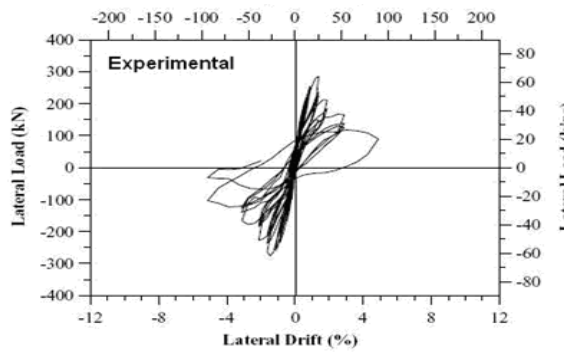**TABLE 2. Parameters used in the Proposed Model for Experimental Specimens.**

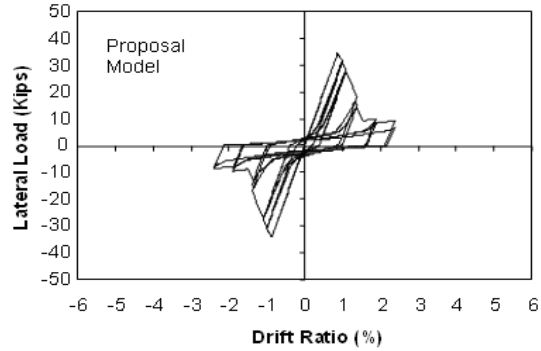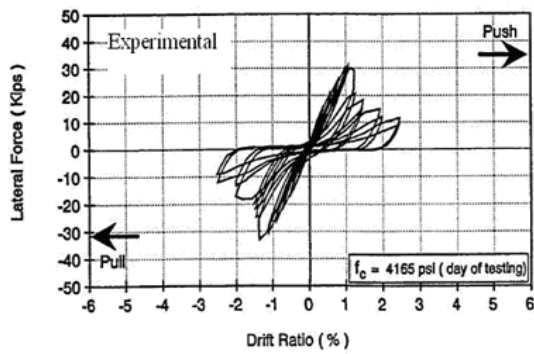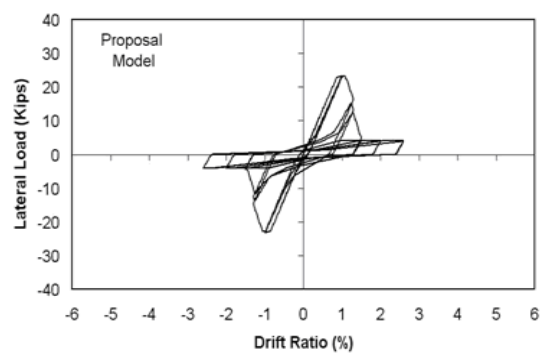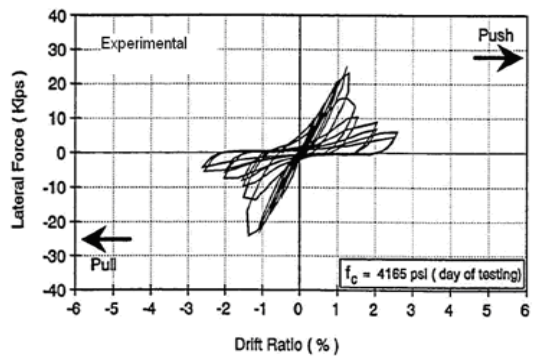| Researcher	Column	$f_{s,max}/f_y$	f_r/f_y	ε_s	ε_r
Aboutaha	FC4	0.79	0.25	0.0039	0.022
	FC14	0.95	0.26	0.0043	0.022
	FC15	0.95	0.20	0.0043	0.022
Melek	S10MI	0.78	0.26	0.0042	0.022
	S20MI	0.78	0.26	0.0042	0.022
	S30MI	0.78	0.26	0.0042	0.022
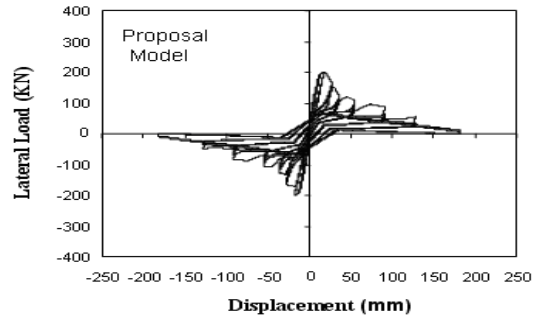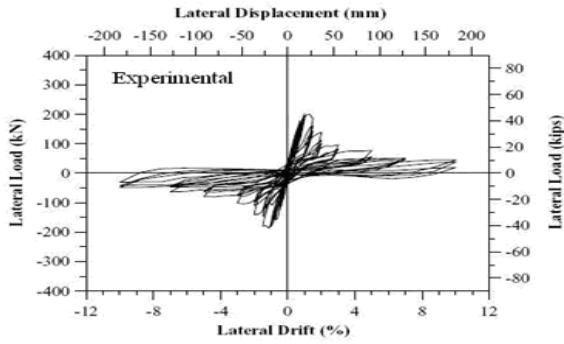
(a)
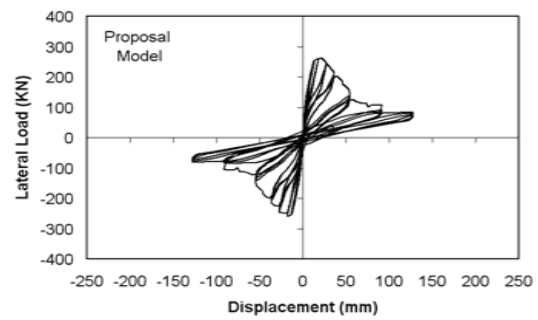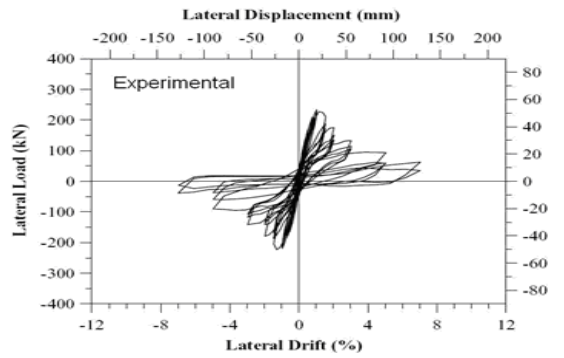
(b)

(c)
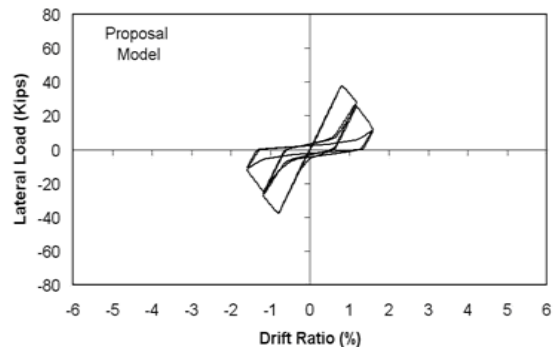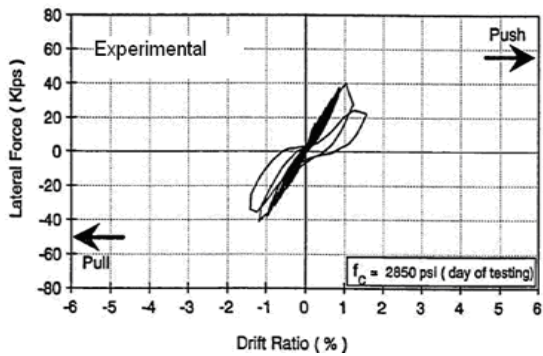
(d)

(e)

(f)

Figure 3. Comparison of proposed model and experimental results for columns with lap-splice;
(a-c) Specimens S10MI, S20MI and S30MI of Melek, et al [10];
(d-f) Specimens FC4, FC14, FC15 of Aboutaha, et al [11].

## 5. FRAGILITY CURVE DEVELOPMENT FOR A ONE BAY-ONE STORY CONCRETE FRAME

In this section, it is intended to develop the fragility curves of a one bay-one story concrete frame, where the main emphasis is on the effect of bar slip in the lap splice. The structure is designed for seismic load of 29 KN, satisfying ACI 318-08 requirements for ordinary moment resisting frames (OMRF). The model considers the slab contribution in stiffness and strength of the beam. The properties of the frame elements used in the analyses are described in Table 3. Note that frictional bar stress in the spliced bars is only 35% of its yield stress. Designing the same frame as intermediate moment resisting frame, not only reduces the longitudinal bar diameter, but also requires tighter spacing for transverse reinforcement. This results in

frictional stresses of about the same value as bars yield stress, which means there will be no strength degradation at large strains.

The fragility curves are generated for life safety and collapse limit states, using the results of incremental dynamic analyses (IDA). For each limit state, fragility curves are evaluated with and without considering the effect of bar slip in columns lap splice. In the deriving of the fragility curves, it is assumed that the IDA results follow a lognormal distribution [30]; this greatly simplifies the derivation procedure. List of ground motions used in the incremental dynamic analyses are given in Table 4.

The recommended drift limit for life safety and collapse limit states in FEMA 356 provisions are 2 and 4 percentages, respectively. Figure 5 depicts the results of incremental dynamic analysis with and without bar slip in lap splice, where spectral

(a)

(b)

(c)

(d)
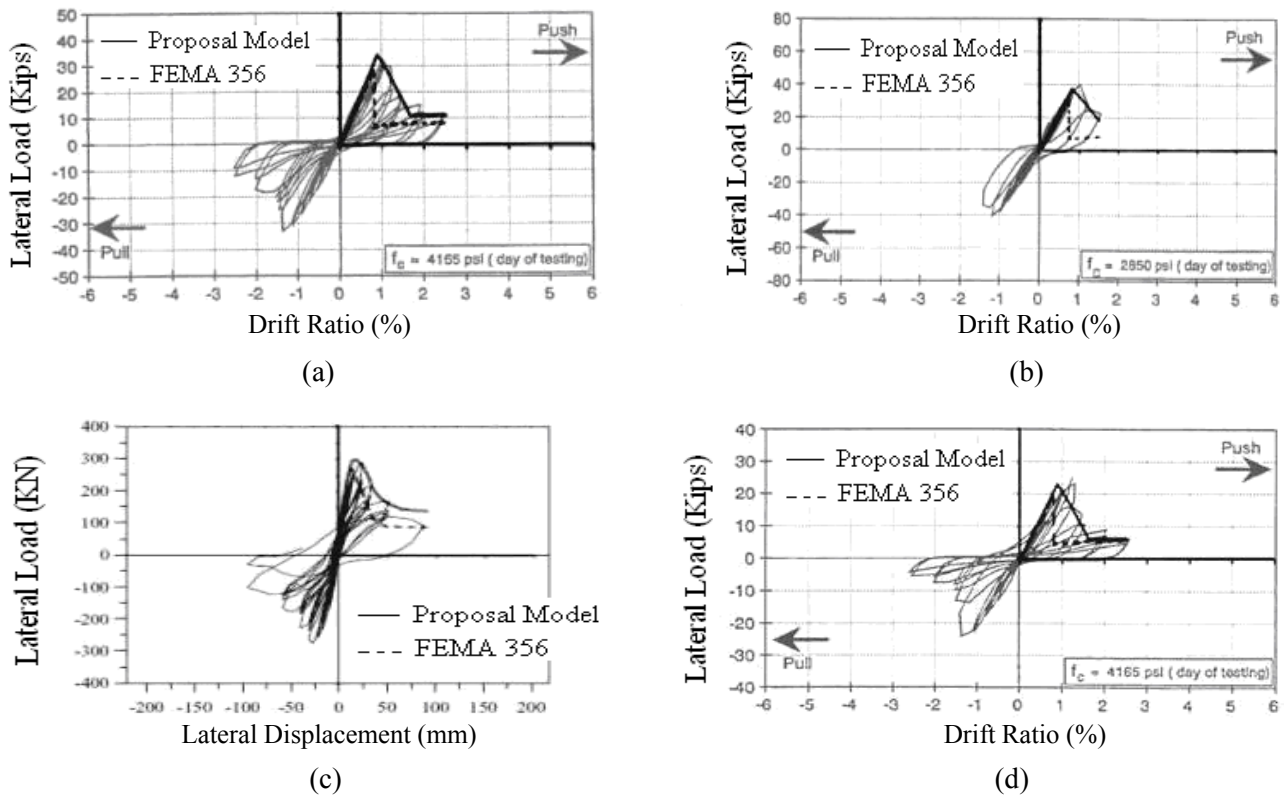
Figure 4. Comparison of the results of the proposed model and FEMA 356 model against experimental resuts: (a) FC14, (b) FC4, (c) S30MI and (d) FC15.

TABLE 3. Column and Beam Properties of the Frame.

Columns (Length 3.2 m)								
Dim (mm)	Long Bars	Trans Bars	Splice Length (mm)	f'_c (Mpa)	f_y (Mpa)	f_{yh} (Mpa)	$\dfrac{f_{s,max}}{f_y}$	$\dfrac{f_r}{f_y}$
300x300	8Φ20	2Φ10at300	580	21	400	300	0.99	0.35

Beam (Span 5 m)						
Dim (mm)	Mid span Bars		Support Bars		DL (KN/m)	LL (KN/m)
	Top	Bot	Top	Bot		
300x300	2Φ16	3Φ18	3Φ20	2Φ16	28	8

acceleration at first period of structure is used as ground motion intensity measure and story drift is used as damage measure. The figure denotes the mean value of response for seventeen earthquakes considered in the analyses and mean plus/minus of the variance. As can be seen, effect of bar slip in the columns lap splice is significant. At the same time, with the increase of ground motion intensity, the variance in the response increases, that is an indication of larger uncertainty in the determination of response parameters for more intensive ground motions.

TABLE 4. List of Ground Motions used in the Incremental Dynamic Analysis.

| No.	Earthquake	Station	Component	Moment Magnitude	Distance (km)
1	Friuli-Italy 1976	Codroipo	0	6.5	33.4
2	Cape Mendocino	Eureka	0	7.01	41.97
3	Cape Mendocino	Eureka	90	7.01	41.97
4	Imperial Valley	El Centro	225	6.5	18.2
5	Imperial Valley	Calipatria	315	6.5	23.17
6	Loma Prieta	Gilory	0	6.9	16.1
7	Loma Prieta	Palo Alto	90	6.9	36.3
8	Northridge	La Saturn	20	6.7	30
9	Northridge	Hollywood	90	6.7	25.5
10	Chi Chi	CWB Chy002	0	7.62	24.21
11	Chi Chi	CWB CHY004	90	7.62	24.21
12	Duzce	Yarimca	0	7.14	97.53
13	Duzce	Yarimca	90	7.14	97.53
14	Kocaeli	Ambarli	0	7.51	69.62
15	Kocaeli	Ambarli	90	7.51	69.62
16	Kobe	Kakogava	0	6.9	22.5
17	Kobe	Kakogava	90	6.9	22.5

Figures 6 and 7 show the fragility curves for the probability of exceeding life safety and collapse limit states, where the effect of bar slip on the fragility curve is more pronounced for collapse limit state. This is an indication of the fact that modeling of bar slip becomes more important for more intensive ground motions. As discussed above, by design of the same structure as IMRF, the effect of bar slip on the response becomes minimal. This is an indication of the adverse effect of lap splice on the response of the structure, and show that at least for structure under consideration providing confinement for concrete in splice length is an important consideration for survival of the structure in the case of intense ground motions.

6. CONCLUSION

A model for evaluating the cyclic behavior of bar slip in lap-spliced columns is proposed. The model is capable of simulating the effects of longitudinal and transverse reinforcement arrangements and their diameter and yield strengths, and considers the degrading and pinching effects due to bar slip in cyclic loading. Comparison with experimental results indicates that the proposed model can predict properly the maximum strength and displacement and post-peak behavior of the tested specimen. At the end, fragility curves for life safety and collapse limit states are developed for a typical one bay-one story frame designed as ordinary moment resisting frame. Results show that bar slip in lap splice has great impact on the response of the structure, and ignoring the effect of the possible bar slip results in the substantial underestimation of probability of exceeding the collapse limit state. At the same time fragility curves for life safety limit state is not affected so hard by the bar slip in lap-spliced column.
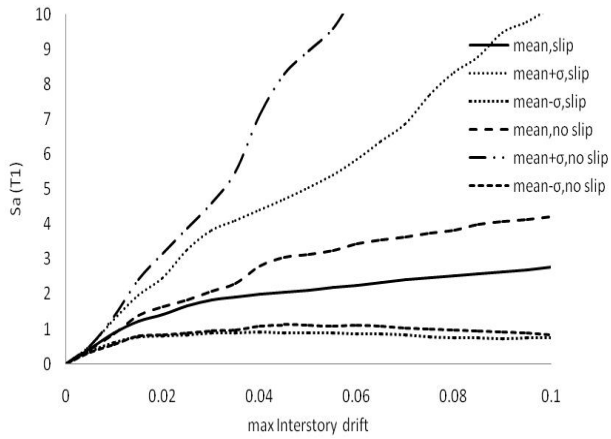
Figure 5. Results of incremental dynamic analysis with and without taking into account for bar slip in the lap splice.
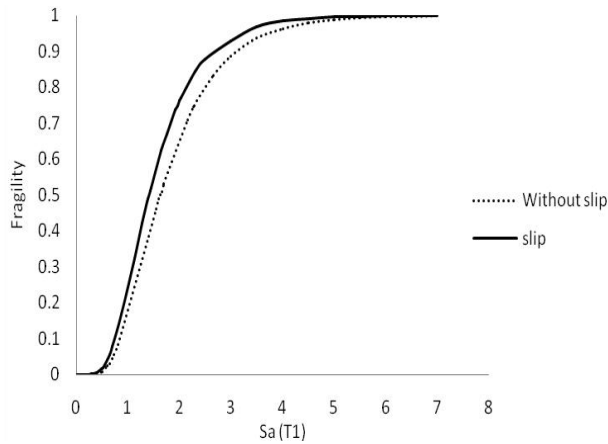
Figure 6. Fragility curve for life safety limit state.
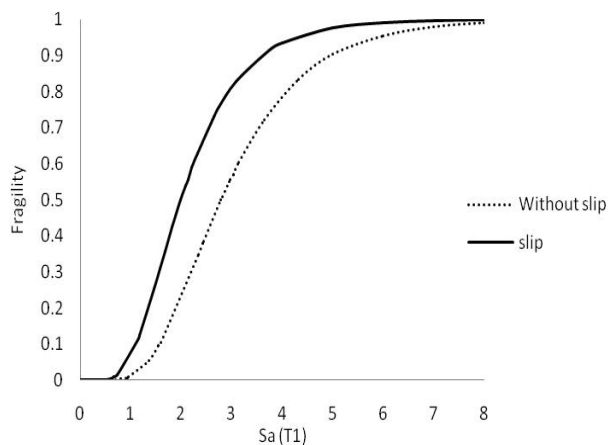
Figure 7. Fragility curve for collapse limit state.

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