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EFFECT OF CONNECTION FRACTURE DISPERSION ON SEISMIC DEMANDS OF WELDED STEEL MOMENT FRAMES

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ABSTRACT

Appropriate modeling of connections behavior is needed for seismic evaluation of structures. However, large dispersion is observed in welded steel moment frame connections performance during recent earthquakes and experimental tests. In this study, the effect of dispersion in pre-Northridge connections characteristics is investigated. Two mid-rise steel moment frames are designated. Various types of fractures in beam-column connections with various configurations are modeled. Sensitivity study is carried out on parameters affecting the structural response for two probability levels by implementation of nonlinear dynamic analysis. Results show that variation in connection characteristics moderately affects the displacement demands of structures.

Keywords: Degradation; hysteresis loop; welded connection; IDASS; pre-northridge; connection fracture

1. INTRODUCTION

The 1994 Northridge and a year later the Kobe earthquakes revealed several deficiencies in steel moment connections. One side, widespread losses and the other side existence of numerous older vulnerable steel moment frames, forced engineering society to implement new approaches for design and seismic evaluation purposes. To do this, several questions about modes of failure and parameters affecting the performance of connections, must be answered first. Lee and Foutch [1] investigated performance of steel moment frames with Pre-Northridge connections. They used a reliability framework to evaluate the effect of

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brittle connection, panel zone and interior gravity frames for 2/50 and 50/50 probability levels. Foutch and Yun [2] used two groups of models for seismic evaluation. First, centerline models with nonlinear beam-columns and panel zones and second, Pre-Northridge connections. Results were compared with elastic models which are used for steel frames design. Yun et al. [3] proposed a performance evaluation approach for steel moment frames based on reliability theory using nonlinear dynamic analysis.

Roeder [4] investigated variations in yield mechanisms and failure modes for pre-Northridge connections. Several connection types were evaluated and strategies were proposed to improve the seismic performance of connections. Ibarra et al. [5] proposed hysteresis models for steel and reinforced concrete members. They used bilinear, peak-oriented and pinching models to calculate inelastic response of SDOF systems.

Since the quality of connection fracture and affecting parameters are not clearly known, this study attempts to investigate the effect of various possible types of brittle connection behavior on the seismic performance of mid-rise buildings. two model structures were designed in accordance with Iranian design codes. Fracture in both beams and column members were modeled and finally the sensitivity of structural response to hysteresis parameters was investigated.

2. HYSTERETIC BEHAVIOR MODELING

A prevalent failure mode in Pre-Northridge connections is the fracture of the beam flange weld, as shown in Figure 1. Fracture in column is also probable. In this case, fracture is initiated in the beam flange and is propagated toward the column flange or web [6]. When connection fractures, moment strength of the connection, M_{red}, drops to a fraction (20% to 30%) of its plastic moment capacity, M_p. For column fracture, moment strength drop is more expected. Since, the residual moment is about 10% or 20% of the column plastic moment capacity [7]. Connection fracture may occurs before nominal plastic capacity of member called "early fracture" (Figure 3) or at a pre-specified rotation, θ_f, (Figure 4). Reduction in moment capacity is observed when the crack is opened. Initial strength is reversed when crack is closed again [8]. Figure 2 depicts an experimental hysteretic behavior of a pre-Northridge connection.

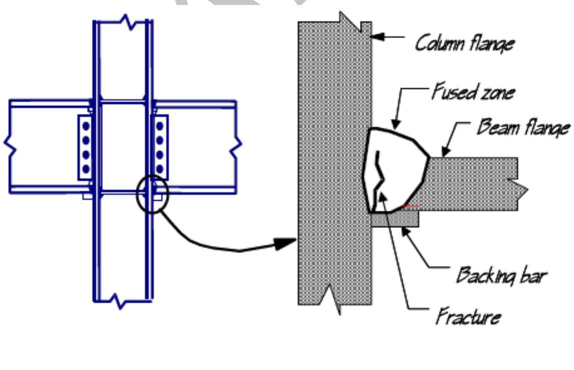


Figure 1. Brittle fracture in the heat affected zone [9]

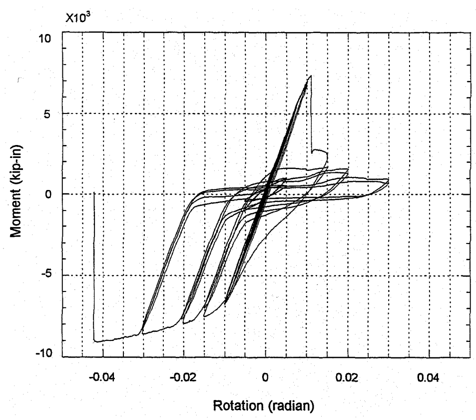


Figure 2. Typical hysteresis rule for a pre-Northridge connection [10]

Dispersion of test results due to different types of fracture and also lack of sufficient knowledge about quality of fracture makes attentive modeling of connection behavior difficult. Therefore, analytical models for connection fracture are simplified to some extent. For this reason, analytical models with ability to model post fracture behavior of a connection are scarce [11-14]. Moreover, locations of connection fracture in structure are randomly selected [15].

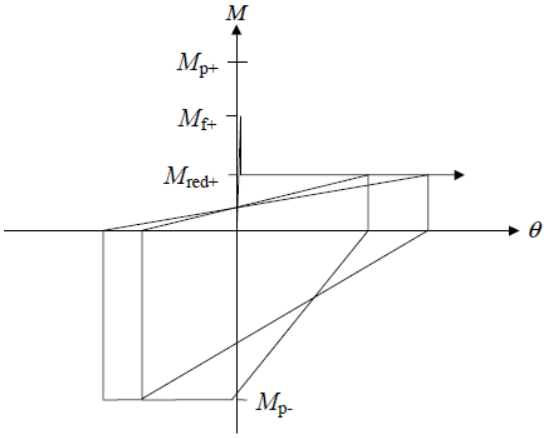


Figure 3. Early fracture of a connection [16]

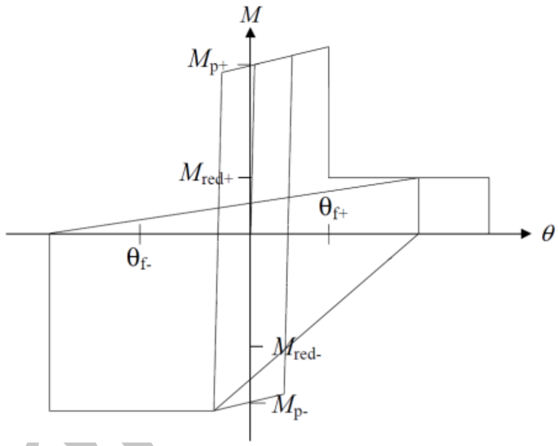


Figure 4. Fracture at pre-specified rotation [16]

3. MODELS AND GROUND MOTION RECORDS

Two 5-story and 9-story special steel moment frames designed in accordance with Iranian seismic code [17] and Iranian steel design code [18]. Models have stories with 3.2m height and bays with 4.0m length, located on area with very high seismicity and soil type III [17]. Figure 5 depicts a view of structures in plan and in elevation. Buildings are square in plan. Thus 2D frames are selected for analysis as shown in figure 5. It is supposed that steel's yield stress is 2400 *Kg/cm²*.floor's dead and live load are 700 *Kg/m²* and 200 *Kg/m²*.

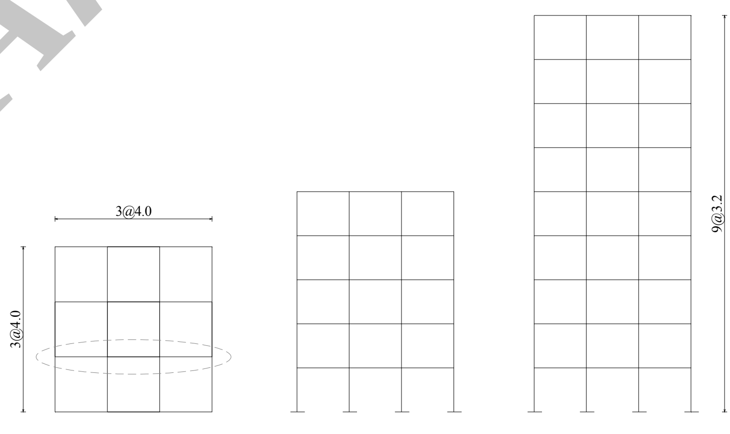


Figure 5. Selected frames view in plan and elevation

In addition, a set of twenty ground motion records were selected. All recorded on soil type III and include no directivity effect. The records are selected from the PEER Center Ground Motion Database [19] as listed in Table 1. For 10%/50 probability level, the ground motions were scaled to coincide to the design spectrum in the range of 0.2T to 1.5T. The design spectrum is scaled by a factor of 1.5 to produce the 2%/50 hazard spectrum [20]. Figure 6 shows the scaled ground motions for 10%/50 probability level.

Table 1: Selected records for dynamic analysis

Record ID	Event	Year	M[1]	Station	R[2] (km)	Soil[3]	Mechanism	PGA (g)
LP89agw	Loma Prieta	1989	6.9	Agnews State Hospital	28.2	D	reverse-oblique	0.172
LP89cap	Loma Prieta	1989	6.9	Capitola	14.5	D	reverse-oblique	0.443
LP89g03	Loma Prieta	1989	6.9	Gilroy Array #3	14.4	D	reverse-oblique	0.367
LP89g04	Loma Prieta	1989	6.9	Gilroy Array #4	16.1	D	reverse-oblique	0.212
LP89gmr	Loma Prieta	1989	6.9	Gilroy Array #7	24.2	D	reverse-oblique	0.226
LP89hch	Loma Prieta	1989	6.9	Hollister City Hall	28.2	D	reverse-oblique	0.247
LP89hda	Loma Prieta	1989	6.9	Hollister Differential Array	25.8	D	reverse-oblique	0.279
LP89svl	Loma Prieta	1989	6.9	Sunnyvale - Colton Ave.	28.8	D	reverse-oblique	0.207
NR94cnp	Northridge	1994	6.7	Canoga Park - Topanga Can.	15.8	D	reverse-slip	0.420
NR94far	Northridge	1994	6.7	LA - N Faring Rd.	23.9	D	reverse-slip	0.273
NR94fle	Northridge	1994	6.7	LA - Fletcher Dr.	29.5	D	reverse-slip	0.240
NR94glp	Northridge	1994	6.7	Glendale - Las Palmas	25.4	D	reverse-slip	0.206
NR94nya	Northridge	1994	6.7	La Crescenta-New York	22.3	D	reverse-slip	0.159
NR94stc	Northridge	1994	6.7	Northridge - 17645 Saticoy St.	13.3	D	reverse-slip	0.368
SF71pel	San Fernando	1971	6.6	LA - Hollywood Stor Lot	21.2	D	reverse-slip	0.174
SH87icc	Superstition Hills	1987	6.7	El Centro Imp. Co. Cent	13.9	D	strike-slip	0.258
SH87bra	Superstition Hills	1987	6.7	Brawley	18.2	D	strike-slip	0.156
SH87icc	Superstition Hills	1987	6.7	El Centro Imp. Co. Cent	13.9	D	strike-slip	0.358
SH87pls	Superstition Hills	1987	6.7	Plaster City	21.0	D	strike-slip	0.186
SH87wsm	Superstition Hills	1987	6.7	Westmorland Fire Station	13.3	D	strike-slip	0.172

[1]moment magnitude [2]closest distance to fault rupture [3]NEHRP site class

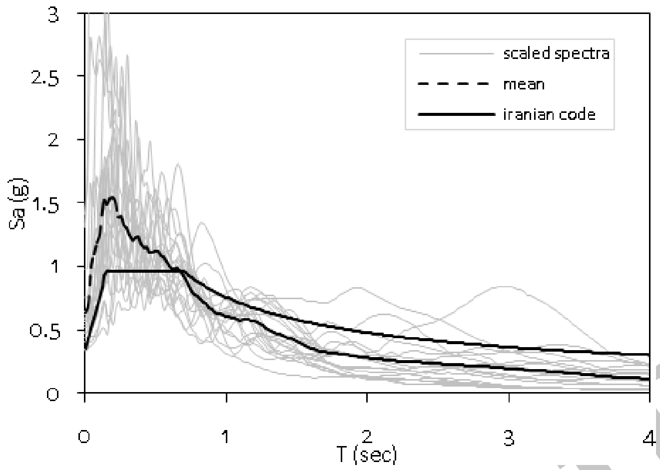


Figure 6. Scaled ground motion records

4. MODELING AND SENSITIVITY STUDY

4.1 Modeling assumptions

Nonlinear dynamic analysis was employed to study the connection fracture effects. Brittle and ductile connections are compared using story drifts as response parameter of interest. Due to random nature of connection fracture, sensitivity studies were carried out to make a better understanding of the effect of various parameters on response values. To model the brittle connection, results of Sac Joint Venture experiments on moment connections [21-22] were implemented. For modeling and analysis IDASS [23] program was used. IDASS is a modification of IDARC [24] which is able to model behavior of ductile and brittle connections [25]. Centerline models with no panel zone effect were considered. Every connection is assumed to experience fracture in a pre-defined rotation. For comparison purpose, a ductile bilinear connection behavior was also modeled.

4.2 Sensitivity study

In attempt to investigate the sensitivity of responses to fracture parameters, a base model was defined first. This model is assumed to be the most plausible to experience fracture. To account for sensitivity of drift responses to hysteresis parameters, each parameter is separately changed. To estimate drift demands, the maximum peak story drift angle ($\theta_{\max}$) and the average peak story drift angle (θ_{ave}) were used for both 10%/50 and 2%/50 probability levels. In addition, peak story drift angle demands were used to show displacement demand in structure height. The "median" values for drift demands are calculated according to

$$\text{"median"} \equiv \exp\left[\frac{1}{n}\sum_{i=1}^{n}\ln(x_i)\right] \tag{1}$$

Also 1-sigma level is calculated as

$$"\text{1-sigma}" \equiv "\text{median}"\exp\left[\sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left[\ln(x_i)-\ln("\text{median}")\right]^2}\right] \qquad (2)$$

5. FRACTURE IN BEAMS

5.1 Base model

It is assumed that 25% of beam connections fracture before reaching plastic moment capacity based on Maison and Bonowitz [26] investigations. For each ground motion, location of connection fractures was selected randomly. The other 75% of connections experience fracture in θ_{f+}=0.015. Hysteresis parameters for the base model are depicted in Table 2. Symbols are shown in Figures 3 and 4.

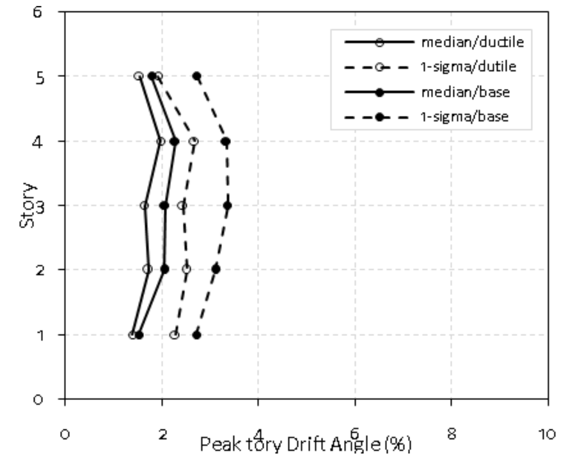
Table 2: Base model hysteresis parameters

Percentage	M_f/M_p	θ_f	θ_{f-}	M_{red}/M_p
25%	0.75	-	∞	0.3
75%	-	0.015	∞	0.3

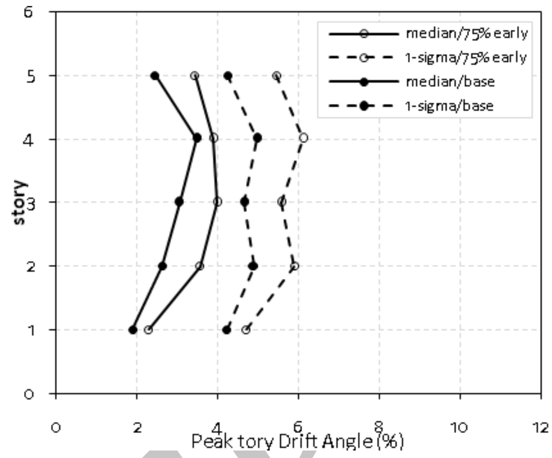
The median and 1-sigma levels of θ_{max} and θ_{ave} for the 10%/50 and 2%/50 probability levels are listed in Table 3. In addition, increases in θ_{max} and θ_{ave} from the early fracturing case to the brittle base case are shown in the table. It is evident that connection fracture causes drift demands to considerably increase up to about 35% for 5-story model and 60% for 9-story models.

Table 3: 5-Story Model with ductile and brittle connections

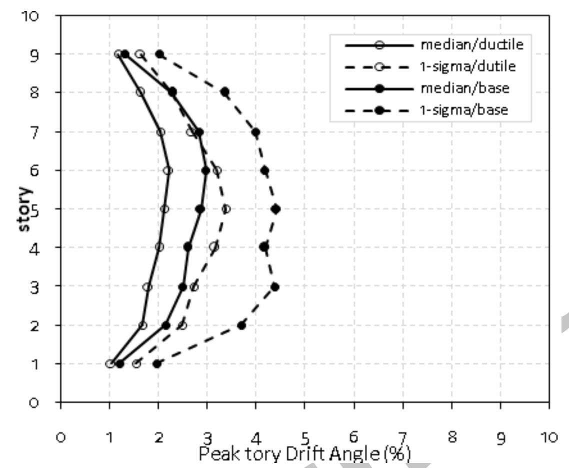
			5-story model			**9-story model**		
			Ductile (%)	**Brittle base (%)**	**Increase (%)**	**Ductile (%)**	**Brittle base (%)**	**Increase (%)**
θ_{max}	10/50	Median	2.11	2.58	22	2.43	3.52	45
		1-sigma	2.84	3.81	34	3.41	4.96	45
	2/50	Median	3.58	4.16	16	3.27	5.12	57
		1-sigma	4.43	5.72	29	4.48	6.97	56
θ_{ave}	10/50	Median	1.68	1.99	18	1.76	2.38	35
		1-sigma	2.30	2.96	29	2.47	3.32	34
	2/50	Median	2.47	2.87	16	2.37	3.63	53
		1-sigma	3.58	4.25	19	3.32	5.04	52



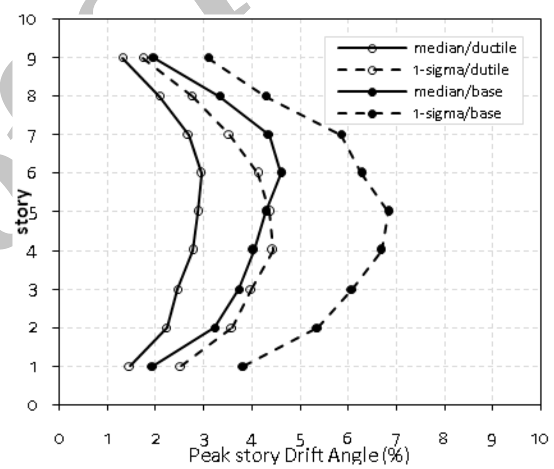
a) 9-story model (10% probability level)



b) 9-story model (2% probability level)



c) 9-story model (10% probability level)



d) 9-story model (2% probability level)

Figure 7. Peak inter-story drift angles for the for base case

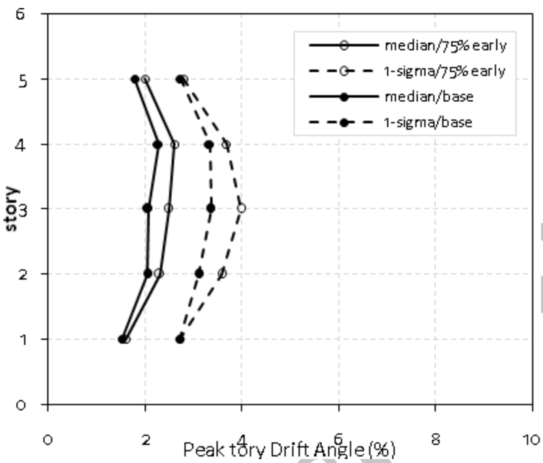
Figure 7 shows that by increase in height the structure is more affected by beam fracture. This increase is more obvious for θ_{max} compared to θ_{ave}, representing the fact that local plastic rotations for brittle connection are increased. Variation of story drifts in brittle cases is more. Moreover, increase in drift demands is more observable for 1-sigma level especially for medial stories.

5.2 Early fracture

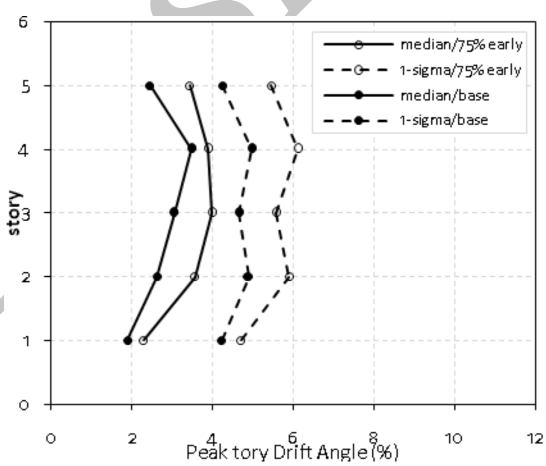
To study the effect of early fracture on the structure response, a new model was considered adopted from the base brittle case with the difference that 75% of beam connections are capable to experience early fracture. Location of these connections was selected randomly. Table 4 compares drift demands for these two cases.

Table 4: Comparison of models with different amount of early fracturing connections

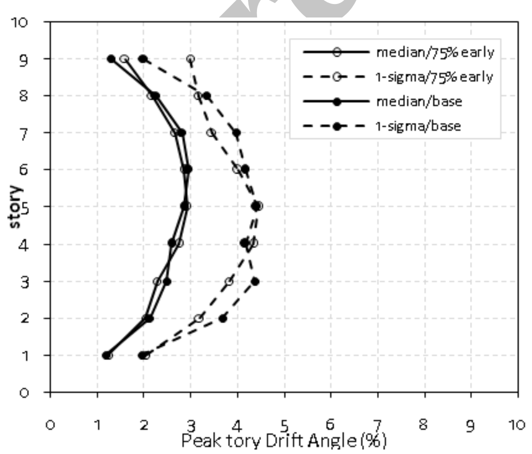
			5-story model			9-story model		
			25% early	**75% early**	**Increase (%)**	**25% early**	**75% early**	**Increase (%)**
θ_{max}	10/50	Median	2.54	2.83	10	3.52	3.52	0
		1-sigma	3.81	4.02	6	4.96	5.23	5
	2/50	Median	4.16	4.25	2	5.12	4.35	-
		1-sigma	5.72	6.32	10	6.97	6.02	-
θ_{ave}	10/50	Median	1.99	2.24	13	2.38	2.36	-1
		1-sigma	2.96	3.26	10	3.27	3.37	3
	2/50	Median	2.87	3.30	15	3.63	3.04	-
		1-sigma	4.25	4.76	12	5.04	4.35	-



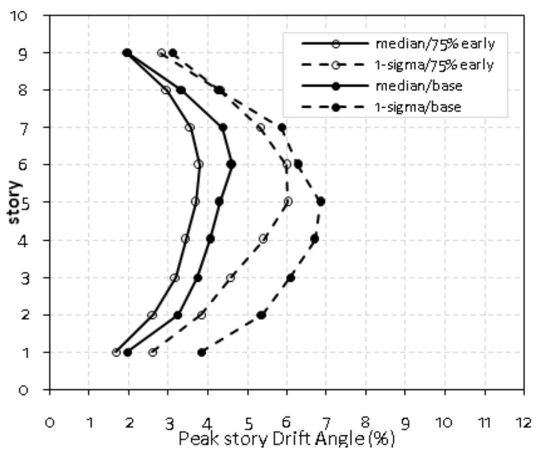
a) 5-story model (10% probability level)



b) 5-story model (2% probability level)



c) 9-story model (10% probability level)



d) 9-story model (2% probability level)

Figure 8. peak inter-story drift angles for 75% early fracturing connections

It is evident that the number of early fractures has no considerable effect on drift responses. However, for higher probability levels the effect of early fractures is more sensible. It is notable that 9-story model experienced collapse at 2%/50 hazard level which prevented to calculate counted statistics.

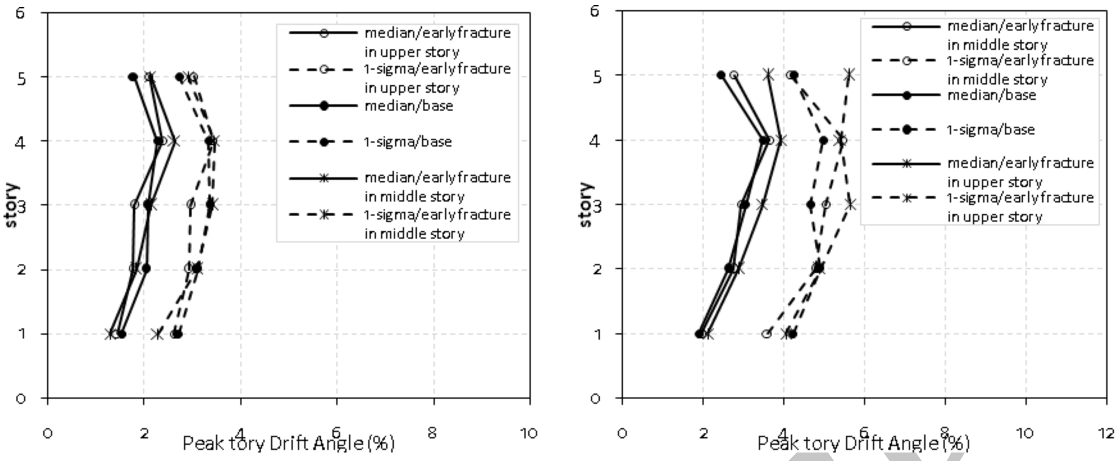
5.3 Distribution of early fracture

For the ductile case maximum drift is observed in middle (for 5 and 9-story models) and upper stories (for 5-story model). Thus, to investigate the worst case affecting the drift responses, two other patterns were used including: Early fractures concentrated in upper stories and concentrated in middle stories.

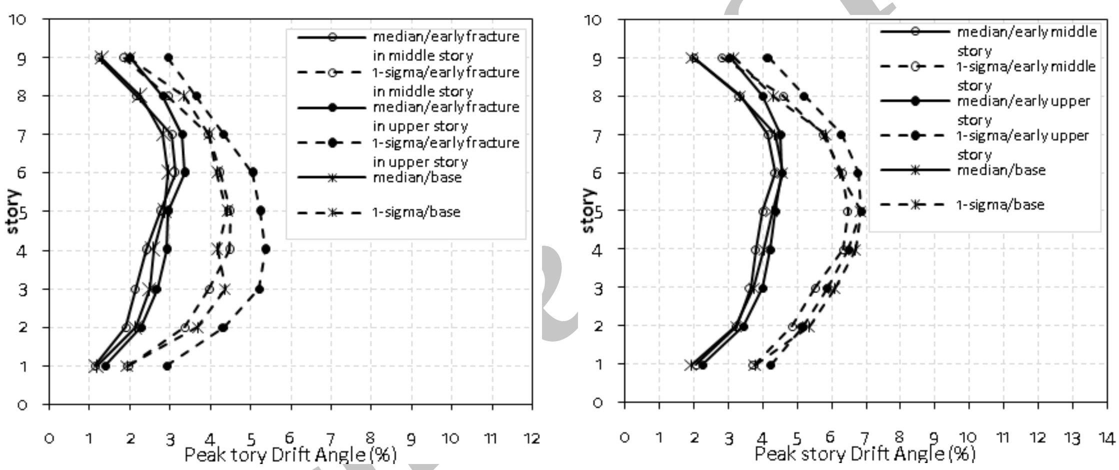
Table 5: Comparison of models with different locations of early fracturing connections

			5-story model				**9-story model**			
			early fracture in		**increase to base**		**early fracture in**		**increase to base**	
			upper	**middle**	**upper**	**middle**	**upper**	**middle**	**upper**	**middle**
θ_{max}	10/50	Median	2.74	2.86	6.2	10.9	3.90	3.58	10.8	1.7
		1-sigma	3.96	3.77	3.9	-1.0	5.39	4.73	8.5	-4.8
	2/50	Median	4.11	3.60	-1.2	-13.5	5.15	4.81	0.6	-6.1
		1-sigma	5.56	5.11	-2.8	-10.7	6.85	6.82	-1.6	-2.0
θ_{ave}	10/50	Median	1.97	2.16	-1.0	8.5	2.76	2.32	16.0	-2.5
		1-sigma	2.84	3.11	-4.1	5.1	4.09	3.24	22.8	-2.7
	2/50	Median	3.92	2.97	-14.6	3.5	3.91	3.49	7.4	-4.1
		1-sigma	4.88	4.45	-14.8	4.7	5.45	4.93	8.1	-2.2

By concentrating the early fracturing connections, responses are moderately increased for 5-story building. As expected, for 5-story structure θ_{ave} is increased by locating the early fracture connections to middle stories. This effect is more pronounced for 10% probability level. However, for the case of early fracturing connections located on upper stories, increase in drift responses is not tangible.



a) 5-story model (10% probability level) a) 5-story model (2% probability level)



c) 9-story model (10% probability level) d) 9-story model (2% probability level)

Figure 9. peak inter-story drift angles for different early fracture connections distribution

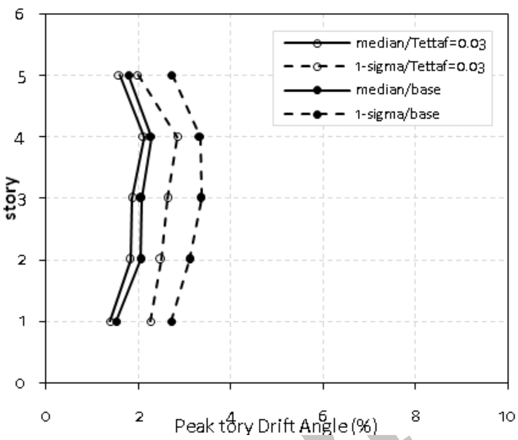
For 9-story model structure, increase in drift demands for 10%/50 ground motion is more than 5-story model structure for the case of early Fractures in upper stories. , location of early fracturing connections (in both patterns of early fractures) has a mild effect on demand measures of the 9-story model structure. For the case that early fracturing connections are assigned to the upper stories, increases (relative to the base case) in the median and 1-sigma level are10-25% for the 10%/50.

5.4 Rotation capacity

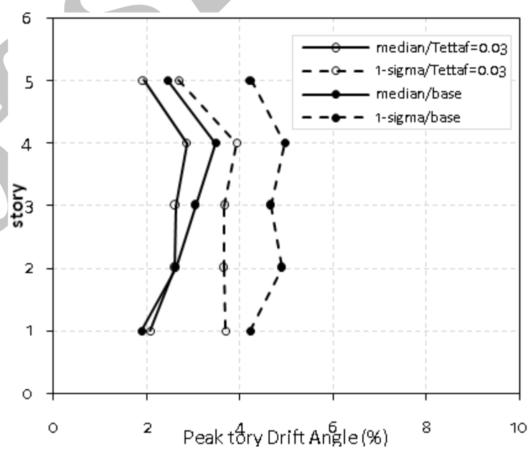
Fracture rotation is changed to study sensitivity of drift demands to fracture rotation values. For this case 0.03 radian rotation is designated as a rational upper limit for fracture rotation.

Table 6: Comparison of models with different fracture rotations

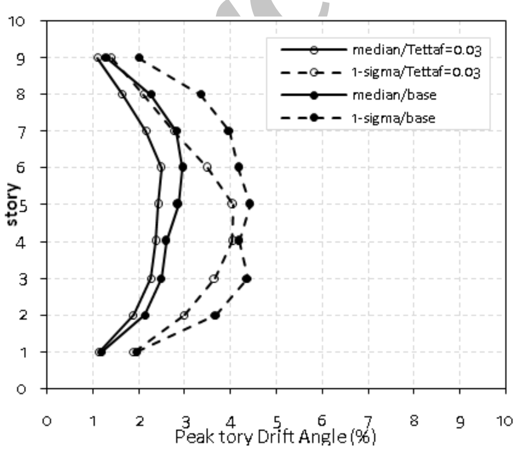
			5-story model			9-story model		
			base	**θ_f=0.03**	**Increase**	**base**	**θ_f=0.03**	**Increase**
θ_{max}	10/50	Median	2.58	2.36	-8.5	3.52	2.62	-25.6
		1-sigma	3.81	3.27	-14.2	4.97	3.84	-22.7
	2/50	Median	4.16	3.25	-21.9	5.12	3.98	-22.3
		1-sigma	5.72	4.64	-18.9	6.96	5.61	-19.4
θ_{ave}	10/50	Median	1.99	1.92	-3.5	2.38	1.98	-16.8
		1-sigma	2.96	2.81	-5.1	3.33	2.86	-14.1
	2/50	Median	2.87	2.49	-16.2	3.64	2.72	-25.3
		1-sigma	4.25	3.35	-21.2	5.04	3.98	-21.0



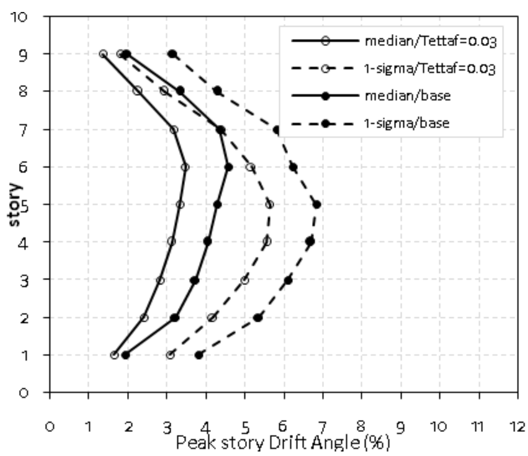
a) 5-story model (10% probability level)



b) 5-story model (2% probability level)



c) 9-story model (10% probability level)



d) 9-story model (2% probability level)

Figure 10. peak inter-story drift angles for θ_f=0.03

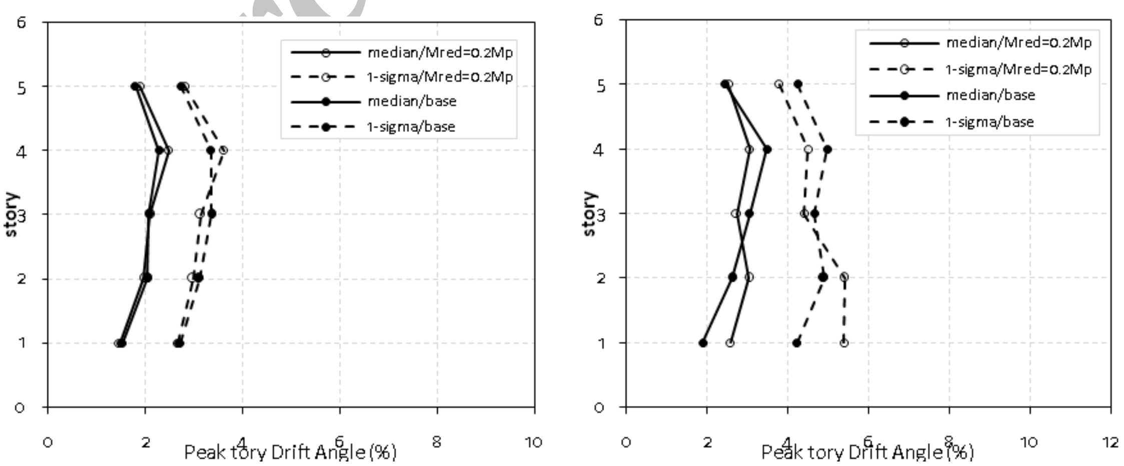
Table 6 lists the statistics for these two patterns of fracture rotation. By increase in the fracture rotation, maximum values of drift responses are decreased. It has relatively significant effect on drift statistics as far as drift statistics decreased up to 25%. For the case of 2%/50, the effect of plastic rotation capacity is more visible. Since, more plastic rotations lead to more connection fractures.

5.4 Residual moment capacity

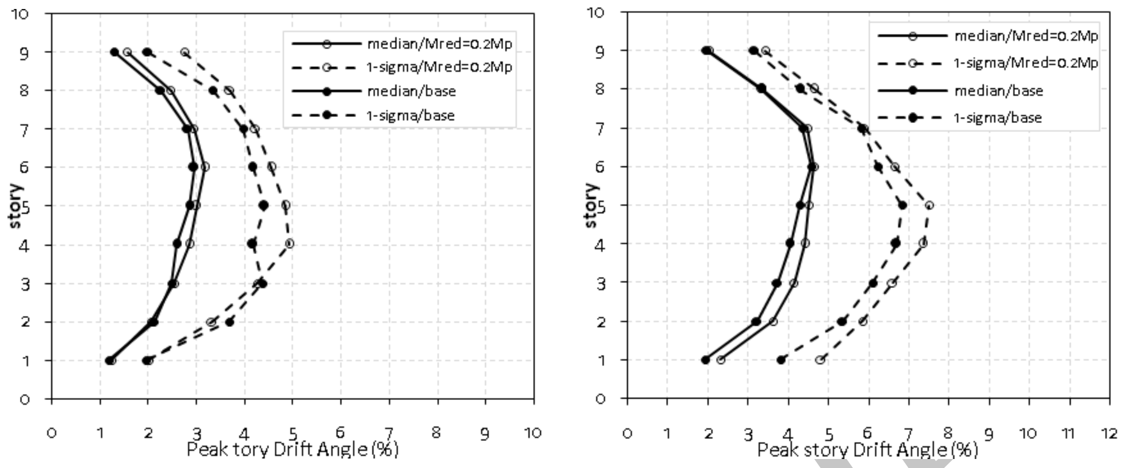
The base case was substituted with a model with residual moment, M_{red}, of 20% of plastic moment, M_p to investigate the effect of residual moment on drift responses. When M_{red}/M_p reduces to 20%, the median and 1-sigma level θ_{max} and θ_{ave} for the 10/50 and 2/50 ground motions in both Model Structure are increased less than 10%. These results indicate that although decrease in *Mred* /*Mp* value increases drift demands as expected, but this effect is not considerable.

Table 7: Comparison of models with different residual moment capacities

			5-story model			9-story model		
			base	**$M_{red} = 0.2M_p$**	**Increase**	**base**	**$M_{red} = 0.2M_p$**	**Increase**
θ_{max}	10/50	Median	2.58	2.78	7.8	3.52	3.76	6.8
		1-sigma	3.81	3.99	4.7	4.97	5.17	4.0
	2/50	Median	4.16	4.10	-1.4	5.12	5.56	8.6
		1-sigma	5.72	5.85	2.3	6.96	7.56	8.6
θ_{ave}	10/50	Median	1.99	2.05	3.0	2.38	2.52	5.9
		1-sigma	2.96	2.84	-4.1	3.33	3.61	8.4
	2/50	Median	2.87	2.98	3.8	3.64	3.90	7.1
		1-sigma	4.25	4.18	-1.6	5.04	5.43	7.7



a) 5-story model (10% probability level) b) 5-story model (2% probability level)



c) 9-story model (10% probability level)

d) 9-story model (2% probability level)

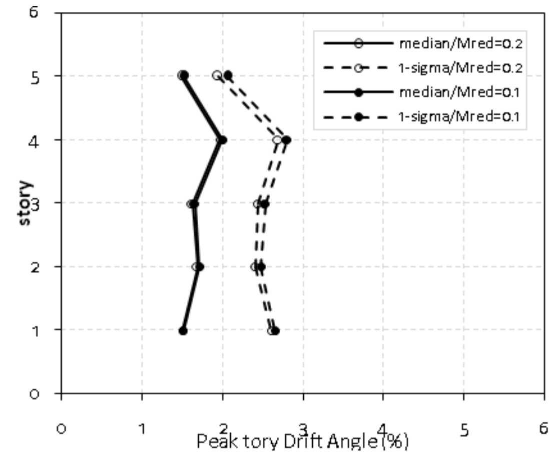
Figure 11. peak inter-story drift angles for 20% residual moment capacity

6. FRACTURE IN COLUMNS

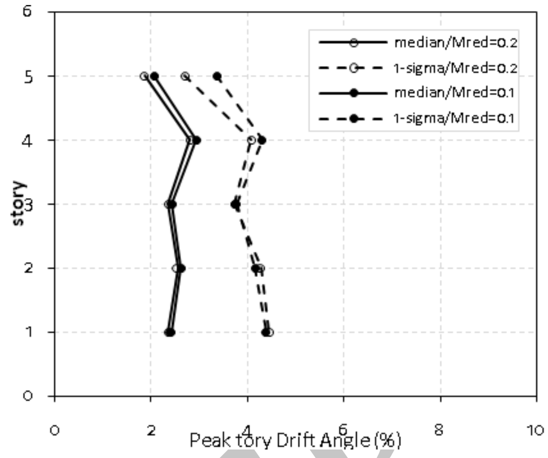
To model column fracture, fracture was allowed to propagate from beam flange to column underneath. Consequently the adjacent beam does not fracture itself. In each structure, it was assumed that columns have 25% probability to experience fracture. Residual moment capacity of columns was taken as 20% of plastic moment capacity.

Table 8: 5-Story Model with different residual moment capacities

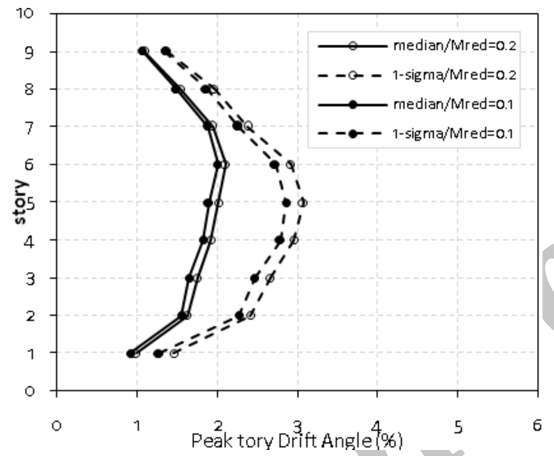
			5-story model				**9-story model**			
			M_{red}=		**Increase for M_{red}=**		**M_{red}=**		**Increase for M_{red}=**	
			0.2	**0.1**	**0.2**	**0.1**	**0.2**	**0.1**	**0.2**	**0.1**
θ_{max}	10/50	Median	2.22	2.27	5	2	2.33	2.23	-4	-4
		1-sigma	3.11	3.24	10	4	3.12	2.94	-9	-6
	2/50	Median	3.19	3.42	-11	7	3.43	3.30	5	-4
		1-sigma	4.67	5.26	5	13	5.07	4.98	13	-2
θ_{ave}	10/50	Median	1.69	1.75	1	4	1.69	1.61	-4	-5
		1-sigma	2.33	2.42	1	4	2.28	2.12	-8	-7
	2/50	Median	2.45	2.54	-1	4	2.44	2.33	3	-5
		1-sigma	3.68	3.84	3	4	3.62	3.43	9	-5



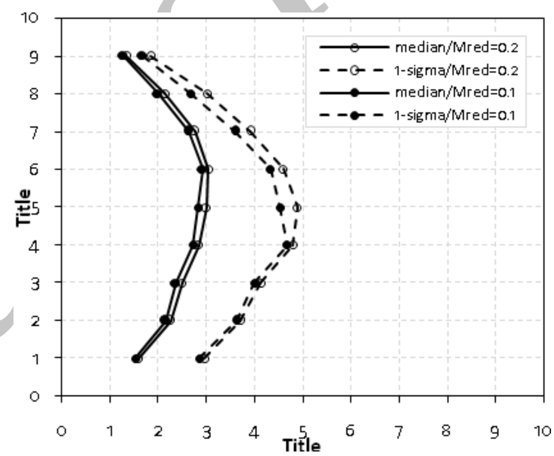
a) 5-story model (10% probability level)



b) 5-story model (2% probability level)



c) 9-story model (10% probability level)



d) 9-story model (2% probability level)

Figure 12. peak inter-story drift angles for column fracture

As can be seen from table 8, column fracture is not important for mid-rise structures. For 5-story structure, increase in drift demands is more observable for the case with lower residual moment. But, this is limited to 13%. For 9-story structure the similar conclusion can be drawn with a difference that the case of M_{red}=0.1 has less effect compared to the case of M_{red}=0.2.

7. CONCLUSIONS

Base on calculations the following conclusions can be drawn:

- A considerable increase in drift demands is observed for the base case compared to the ductile connection case which is more pronounced for the 9-story structure.
- Increase in possibility of early fracture in connections to 75% resulted in drift demands

to increase less than 15%.

- When early fracture connections positioned in upper stories, increase in drift demands is more. The maximum increment in drift statistics is 23% beyond the base model.
- Change in fracture rotation to 0.03 radian led to decrease in drift demands in all of model structures. However this is limited to less than about 20%.
- The largest change in drift statistics was 13% for column fracture which confirms that the effect of column fracture in seismic drift response is not significant.
- In general, hysteresis parameters of fracture are not clearly known. However, these parameters do not have an important role on drift responses.

REFERENCES

1. Lee K., and Foutch DA. Performance prediction and evaluation of steel special moment frames for seismic loads, *SAC Background Document, SAC/BD-00/25*, SAC Joint Venture, Richmond, Calif, 2000.
2. Foutch DA, Yun S. Modeling of steel moment frames for seismic Loads, *Journal of Constructional Steel Research*, **58**(2002) 529–64.
3. Yun S, Hamburger RO, Cornell CA and Foutch DA. Seismic Performance Evaluation for Steel Moment Frames, *Journal of Structural Engineering*, ASCE, No. 4, **128**(2002) 534–45.
4. Roeder CW. Connection Performance for Seismic Design of Steel Moment Frames, *Journal of Structural Engineering*, *ASCE*, No. 4, **128**(2002) 517–25
5. Ibarra L, Medina R, Krawinkler H. Hysteretic models that incorporate strength and stiffness deterioration, *Earthquake Engineering and Structural Dynamics*, No. 12, **34**(2005) 1489–511.
6. FEMA-267. Interim Guidelines, Inspection, Evaluation, Repair, Upgrade and Design of Welded Moment Resisting Steel Structures, *Prepared by the SAC Joint Venture for the Federal Emergency Management Agency*, Washington, DC. Superseded by FEMA 350 to 353, 1995.
7. Anderson JC, Johnston RG, Partridge JE. Post earthquake testing of damaged moment connections, Proceedings of the NEHRP Conference and Workshop on Research on the Northridge, California Earthquake of January 17, 1994, III-B**,** (1998) 478–86.
8. Pinto PE. Probabilistic Methods for Seismic Assessment of Existing Structures, *LESSLOSS Report No. 2007/06*.
9. Krawinkler H, Gupta A, Medina R, Luco N. Loading Histories for Seismic Performance Testing of SMRF Components and Assemblies, *SAC/BD-00/10*, 2000.
10. Goel SC, Stojadinovic B. Test results of Pre-Northridge, simulated filed welding, weld fracture, small plastic rotation. *SAC Test Results*. Ann Arbor, MI: University of Michigan, 1999.
11. Kunnath S. Enhancements to Program IDARC: Modeling Inelastic Behavior of Welded Connections in Steel Moment-resisting Frames, *Building and Fire Research Laboratory National Institute of Standards and technology Gaithersburg*, MD 20899, 1995.
12. Foutch DA, Shi S. *Connection Element, Type 10 for DRAIN-2DX*, University of

Illinois, 1996.
13. Wang CH, Wen YK. Evaluation of pre-Northridge low-rise steel buildings. I: Modeling, *Journal of Structural Engineering,* ASCE, No. 10, **126**(2000) 1160–8.
14. Maison B, Kasai K. Analysis of Northridge Damaged Thirteen-Story WSMF Building, *Earthquake Spectra*, No. 3, **13**(1997) 451−73.
15. FEMA-355C. State of the Art Report on Systems Performance of Steel Moment Frames subject to Earthquake Ground Shaking, *prepared by the SAC Joint Venture for the Federal Emergency Management Agency*, Washington, DC., 2000.
16. Luco N. Probabilistic seismic demand analysis, SMRF connection fractures and near-source effects, PhD Dissertation, Department of CEE, Stanford University, 2003.
17. BHRC Publication No. S-253. Standard No. 2800-05, Third Edition, Iranian Code of Practice for Seismic Resistant Design of Buildings, *Building and Housing Research Center*, 2005.
18. MHUD. Iranian National Building Code, part 10, steel structure design. Tehran (Iran): *Ministry of Housing and Urban Development*, 2006.
19. FEMA-356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, *Federal Emergency Management Agency*, Washington, DC, 2000.
20. Lee K., Foutch DA. Seismic Prformance Evaluation of Pre-Northridge Steel Frame Buildings with Brittle Connections, *Journal of Structural Engineering,* ASCE, No. 4, **128** (2002) 546–56.
21. Yun SY., Foutch DA. Performance prediction and evaluation of low ductility steel moment frames for seismic loads, *SAC Background Document, SAC/BD-00/26*, SAC Joint Venture, Richmond, Calif, 2000.
22. Kunnath SK. IDASS-A Program for Inelastic Damage Analysis of Structural Systems, Technical Report, Department of Civil Engineering, University of Central Florida, 2000.
23. Kunnath SK, Reinhorn AM, Lobo RF. IDARC: A Program for the Inelastic Damage Analysis of Reinforced Concrete Structures, Report No. NCEER-92-0022, National Center for Earthquake Engineering Research, State University of New York at Buffalo, 1992.
24. Gross JL. A connection model for the seismic analysis of welded steel moment frames, *Building and Fire Research Laboratory,* National Institute of Standards and Technology, Gaithersburg, MD 20899, USA.
25. Maison B, Bonowitz D. Opinion paper: how safe are pre-Northridge WSMFs? A case study of the SAC Los Angeles 9-story building, *Earthquake Spectra*, No. 15, **4**(1999) 765−89.