Seismic Demand on Column Splices in Steel Moment Frames

JAY SHEN, THOMAS A. SABOL, BULENT AKBAS and NARATHIP SUTCHIEWCHARN

ABSTRACT

This study addresses seismic demands on column splices in steel moment-resisting frames. A comprehensive nonlinear analytic investigation was undertaken to evaluate the seismic response analysis of 4-, 9- and 20-story moment-resisting frames subjected to an ensemble of 20 strong ground motions. The outcomes of the study include an analysis of the comprehensive seismic demand on the column splice and recommended guidelines for design requirements for reliable moment frame column splices. The study concludes that the demand on the column splice can approach the nominal design strength of the smaller column when the critical beam-to-column connection reaches its expected maximum deformation capacity. It is reasonable that seismic design provisions for the column splices in special and intermediate moment frames require the column splice to develop the flexural strength of the smaller column.

Keywords: column splices, steel moment frames, seismic design

Steel moment frames have been one of the most frequently used seismic force resisting systems in regions of high seismicity. During the 1994 Northridge earthquake, some steel moment frames with welded moment connections suffered damage at or near their beam-to-column joints (FEMA, 2000a). Since then, the structural engineering and steel construction communities have undertaken an extensive research effort, centering on the beam-to-column connection, to investigate the cause of the damage and to improve seismic design, construction, inspection, evaluation and retrofit of steel moment frames. This research resulted in much improved understanding of seismic demand and capacity, as well as improved design requirements for beam-to-column connections in steel moment frames. The research also resulted in enhanced requirements for column splices. For example, current seismic design specification provisions (e.g., AISC 341-10) generally require that column splices in intermediate and special moment frames, when not made using complete-joint-penetration (CJP) welds, be designed to develop the expected flexural strength of the smaller connected column and the shear demand associated with flexural hinging at the top and bottom of a spliced column at a given story assuming a point of inflection at mid-height. Partial-joint-penetration (PJP) welds are currently prohibited in intermediate and special moment frame column splices.

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The following issues appear to play a role in the seismic design practice of column splices:

1. Unless special precautions are taken, welded connections of steel sections subjected to seismic loads are recognized to be more susceptible to brittle fracture than was commonly acknowledged before the 1994 Northridge earthquake. Thus, higher level of filler metal Charpy V-Notch toughness are required for welded column splices in all types of moment frames covered by AISC 341-10.

2. It has been observed that partial-joint-penetration (PJP) welds, when subjected to tensile loads at right angles to the unfused portion of the welded joint, are more susceptible to brittle fracture due to high levels of stress concentration than CJP welds. Thus, AISC 341-10 requires CJP welds in lieu of PJP welds at column splices because of the potential high for flexural or tensile demands consistent with the increased ductility in the improved beam-to-column connection.

3. As suggested by columns bending in double curvature observed in elastic analyses, the demand on column splices, often located in the middle third of the story height, is assumed to be less than that found in the portion of the column directly adjacent to the beam-column joint. It was assumed that the beam-to-column connection would reach its critical limit state before the column splice did.

Nevertheless, the question arises whether the seemingly more conservative column splice seismic design provisions in the current version of AISC 341 can be justified, compared to the column splice requirements in older seismic design provisions. While bolted column splices are permitted, the high strength required by AISC 341-10 often makes them impractical, and the revised column splice provisions often require erection aids necessary to stabilize the column prior to welding and heavy welds to satisfy the specified strength, both of which increase costs. Given the limited detailed research on this topic, a systematic seismic investigation of column splices was conducted to address the question of whether the seismic design provisions requiring development of the expected plastic flexural strength and groove welds at column splices are justified or unnecessarily conservative. A comprehensive study on column splices in steel moment frames was conducted by Shen and Sabol (2008). This paper summarizes the major results related to the seismic demand on the column splices. This demand was evaluated with respect to the demand on the frame system as whole and the demand on the beam-to-column connection in particular so that the influences of uncertainty, such as the type of ground motion and properties of the structural systems, might be properly considered.

STRUCTURES AND GROUND MOTIONS

Design of 4-, 9- and 20-Story Special Moment Frames

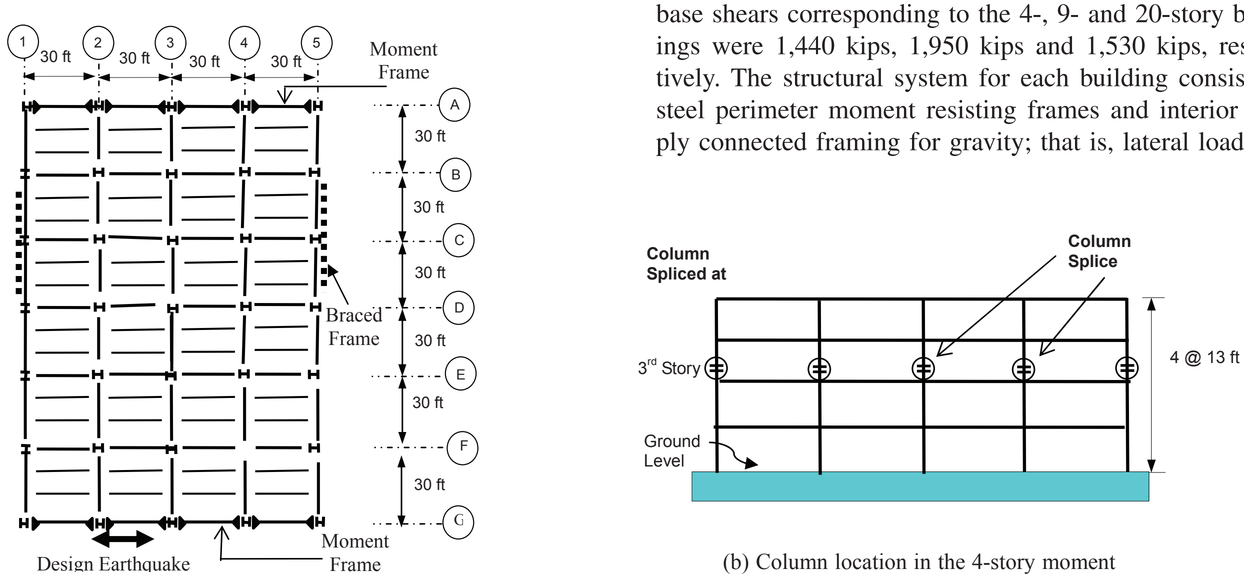
Three typical steel moment frames with heights equal to 4, 9 and 20 stories, representing typical low-, medium-, and high-rise steel buildings (shown in Figures 1, 2 and 3) were designed based on the seismic design requirements in ASCE 7 (2005) and AISC 341 (2005, 2010). These buildings are similar to those developed as part of the FEMA-sponsored steel frame research program conducted following the 1994 Northridge earthquake (FEMA, 2000b). The footprint of each building is symmetrical. As shown in Figure 1, the four-story building has plan dimensions of 120 ft by 180 ft with four 30-ft bays and six 30-ft bays in the two orthogonal directions, respectively, and a typical story height of 13 ft. The columns are assumed to be fixed at the ground level.

The nine-story building has plan dimensions of 150 ft by 150 ft and consists of five bays of framing in both orthogonal directions spaced at 30 ft on center. The building has a basement level (level B1 in Figure 2b). The typical story height is 13 ft except at the ground and B1 levels, where it is 18 ft and 12 ft, respectively (Figure 2b).

The 20-story building has plan dimensions of 100 ft by 120 ft and consists of five 20-ft bays and six 20-ft bays of framing in the two orthogonal directions, respectively. The building has two basements levels (levels B1 and B2). The typical story height is 13 ft except at the ground B1 and B2 levels, where it is 18 ft and 12 ft, respectively (see Figure 3b).

The columns are assumed to be pinned at the lowest basement level for the 9- and 20-story buildings respectively, although they run continuously through the ground level framing. For the 9- and 20-story buildings, concrete foundation walls and surrounding soil are assumed to prevent any significant horizontal displacement of the structure at the ground level, so the seismic base is taken at the ground level.

The buildings were designed for a site in downtown Los Angeles, where S_S is 2.0 g and S_1 is 1.0 g. The perimeter frames of the buildings in the direction of the design earthquake were designed as special moment frames using response modification factor of R = 8. The ASCE 7 (2005) base shears corresponding to the 4-, 9- and 20-story buildings were 1,440 kips, 1,950 kips and 1,530 kips, respectively. The structural system for each building consists of steel perimeter moment resisting frames and interior simply connected framing for gravity; that is, lateral loads are



(a) Plan

(b) Column location in the 4-story moment frame on Lines A and G

Fig. 1. Plan and elevation of the four-story frame.

carried by perimeter frames and interior frames are not explicitly designed to resist seismic loads in the direction of the earthquake and are not included in the analysis. The approximate period equation prescribed in ASCE 7 (2005) was first used to check for strength before the drift requirements were evaluated. As expected, drift requirements governed the design for all three buildings. The member sizes are summarized in Tables 1, 2 and 3, respectively. Braced frames, shown in Figures 1, 2 and 3, are used as the seismic force resisting system in the direction perpendicular to the moment frames.

The location of a column splice is considered to be a factor affecting flexural demand at the splice. With some exceptions, AISC 341 (2005, 2010) requires that column splices be located 4 ft or more away from the beam-column connection. The 4-ft offset is considered to be convenient for field welding and erection and moves the splice closer to the middle of the story height, where the flexure demand is generally thought to be lower than that at the beam-to-column connection. The 4-ft offset is typically interpreted as the distance between the column splice and top of steel girders, but actual locations of column splices may vary to some degree in any given steel building. For example, the User Note in Section D.5a of AISC 341-10 recommends that where possible, splices should be located at least 4 ft above the finished floor elevation to permit installation of perimeter safety cables prior to erection of the next tier and to improve accessibility. On the other hand, Section D.5a(2) of AISC 341-10 also permits a column splice to be located as close to the beam-to-column flange connection as the depth

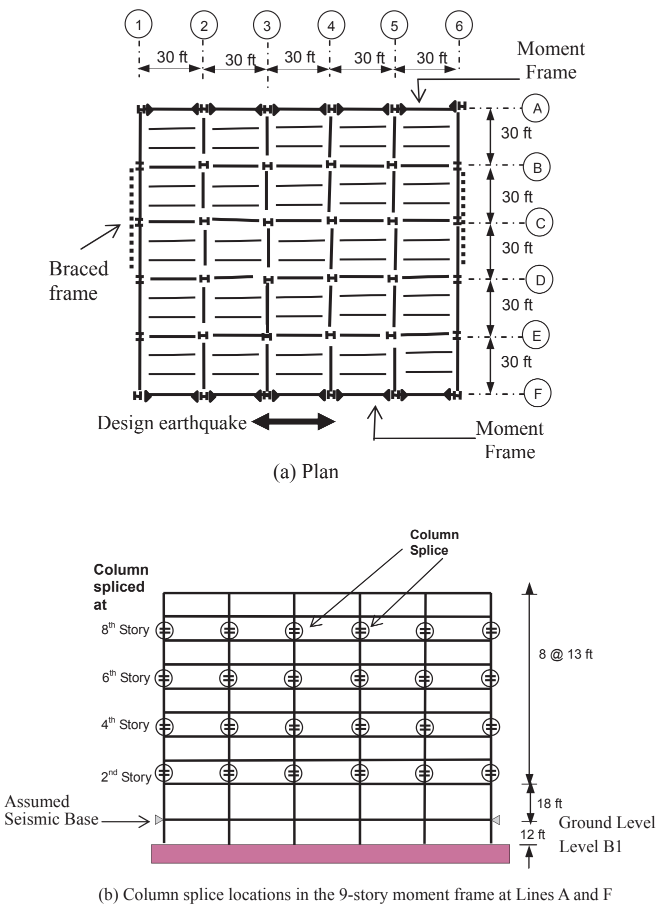


Fig. 2. Plan and elevation of the nine-story frame.

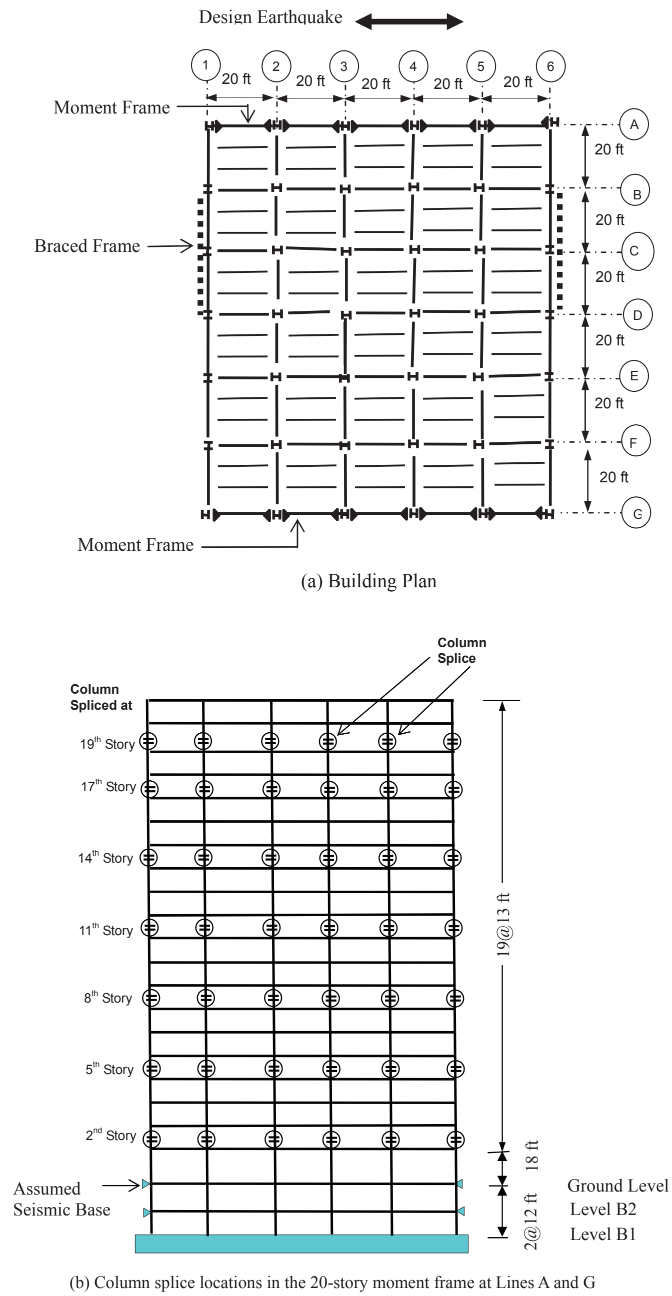


Fig. 3 Plan and elevation of the 20-story frame.

Table 1. Member Sizes of the Four-Story Frame

Level	Exterior Column	Interior Column	Beam
4	W14×257	W14×342	W27×94
3	W14×257	W14×342	W30×148
2	W14×342	W14×426	W30×148
1	W14×342	W14×426	W30×148

Table 2. Member Sizes of the Nine-Story Frame

Level	Exterior Column	Interior Column	Beam
9	W14×257	W14×311	W24×55
8	W14×257	W14×311	W27×94
7	W14×311	W14×426	W30×132
6	W14×311	W14×426	W30×132
5	W14×398	W14×500	W33×141
4	W14×398	W14×500	W33×141
3	W14×455	W14×550	W33×141
2	W14×455	W14×550	W33×141
1	W14×550	W14×730	W36×194
Basement	W14×550	W14×730	W36×194

of the column when the webs and flanges of the splice are connected by complete-joint-penetration groove welds. Two bounding cases, primary (PC) and secondary (SC), were included in this study, as shown in Figure 4. The PC column splice location is 4 ft above the finished floor elevation. The SC column splice location is 4 ft from the beam centerline. These two cases were studied to investigate the impact of shifts in the location of column splices. Nearly all column splices are expected to fall within the locations described by the PC and SC locations. With the 4-ft dimension taken from the top of finished floor, the PC location is considered representative of typical slabs constructed of metal deck and concrete fill and represents the expected upper bound column splice offset. The SC location moves the column splice closer to the beam-column connection (actual distances between column splices and top of steel girders are between $1.20d_c$ and $1.80d_c$ in the three frames studied) and is considered representative of the column splice location permitted by the exception listed in Section D.5a(2) of AISC 341-10. The column splices were located at every second floor of the 9-story frame and at every two to three floors in the 20-story frame. This slight deviation from common practice is not expected to affect the significance of the results, but it did ease the computational burden by reducing the number of column splices that has to be monitored. The columns in the four-story frame were spliced only at its third floor.

Table 3. Member Sizes of the 20-Story Frame

Level	Exterior Column	Interior Column	Beam
20	W24×207	W24×103	W24×55
19	W24×207	W24×103	W27×94
18	W24×207	W24×146	W27×94
17	W24×207	W24×146	W27×94
16	W24×207	W24×192	W27×114
15	W24×207	W24×192	W27×114
14	W24×207	W24×192	W27×114
13	W24×207	W24×279	W30×148
12	W24×207	W24×279	W30×148
11	W24×207	W24×279	W30×148
10	W24×250	W24×306	W30×148
9	W24×250	W24×306	W30×148
8	W24×250	W24×306	W30×148
7	W24×250	W24×335	W30×173
6	W24×250	W24×335	W30×173
5	W24×250	W24×335	W30×173
4	W24×306	W24×370	W30×191
3	W24×306	W24×370	W30×191
2	W24×306	W24×370	W30×191
1	W24×306	W24×370	W30×191
B1	W24×306	W24×370	W30×191
B2	W24×306	W24×370	W30×191

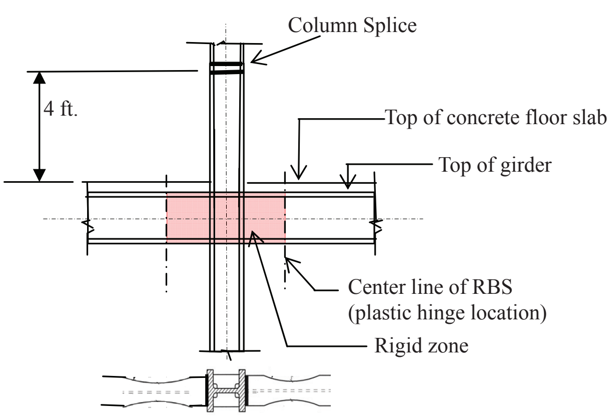
Earthquake Ground Motions and Evaluation Method

The seismic demand on the column splice is significantly affected by selected ground motions included in the analysis, and we have strived to minimize the effects of the uncertainties involved in the ground motion. One approach to minimizing such effects is to take advantage of well established seismic design principles for steel moment frames. The fundamental philosophy in the seismic design of an intermediate or special steel moment frame is to have ductile beam-to-column connections dissipate significant amounts of seismic energy through extensive inelastic deformation so that other structural parts of the frame, including column splices, are not overloaded and remain functional. This helps reduce the potential for collapse during the design earthquake event. In other words, the peak seismic demand on the column splice can be controlled by limiting the maximum rotational capacity of the beam-to-column connection. Therefore, the seismic demand on the column splice ought to be evaluated in comparison with the demand on the entire frame system in general, and on the beam-to-column connection in particular, for any given earthquake ground motion intensity. Thus, we developed a seismic demand evaluation methodology based on a performance-chain concept (Shen et al., 2010) in evaluating the seismic demand on the column splice under meaningful intensities in ground motions. The concept introduced in this study states that the seismic demand on the column splice should be compatible with its intended performance in comparison with that of the frame and beam-to-column connections. Based on this concept, the maximum seismic demand on the column splice is directly related to the seismic demand on the frame as a whole, regardless of the types and intensities of the ground motions selected for the study.

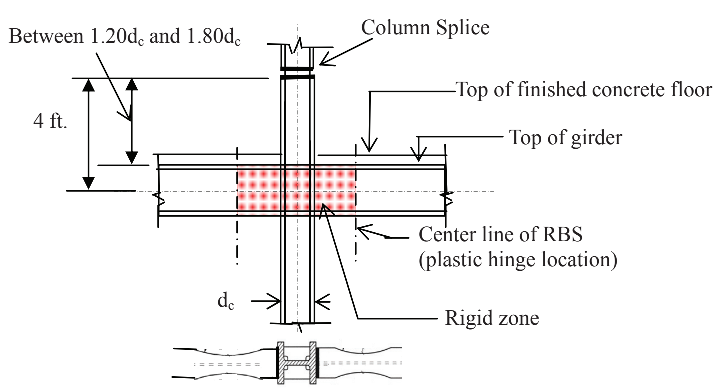
In this study, an ensemble of ground motions was selected so that the seismic response of each of the three frames would range from moderate to severe and the seismic demand on the column splice would be evaluated based on the response of the frames. A total of 20 ground motion records, identified as LA21 to LA40, were used. This set of ground motions were used in a Federal Emergency Management Agency (FEMA)–sponsored research project on steel moment frames damaged in the 1994 Northridge earthquake. This set of ground motions was identified as having a 2% probability of exceedance in 50 years by SAC, a consortium conducting the FEMA-sponsored project. Table 4 provides detailed information on the records. These acceleration time histories were derived from historical recordings or from physical simulations and altered so that their mean response spectrum matches the 1997 National Earthquake Hazards Reduction Program (NEHRP) design spectrum, modified from soil type of S_B–S_C to soil type S_D and having a hazard specified by the 1997 USGS maps (Somerville et al., 1997). Figure 5 summarizes the response spectra of these ground motions.

The seismic response evaluation of the column splice is based on two groups of response parameters reflecting deformation and load: (1) peak story drift ratios and inelastic deformation of the structure, represented by the maximum plastic hinge rotations at beam ends; and (2) peak load demands at the column splice, represented by (a) the maximum bending moment at the splice, M_s, normalized by the plastic moment of the smaller column (on the top of the splice), M_{pt}, and (b) the maximum combination of the normalized bending moment and tensile axial force in the column splice, P_s, normalized by the nominal tensile strength of the smaller column, P_{yt}. The first group of the response parameters provides information about the seismic performance of the frame as a whole for a given ground motion. With the seismic performance of the frame as a reference, the information in the second group is used to evaluate the severity of the demand on the column splice relative to the ground motion with respect to that of the whole frame system. This approach leads to a rational design strength requirement for the column splice within the system, where all components are interrelated, and a desirable hierarchy in the chain of the possible limit states is well defined.

In a special steel moment frame, the beam-to-column connection may well be the most critical component when subjected to strong ground motions, and its seismic behavior has been well documented in other research studies. The demand on the connection, therefore, is considered to be a reliable reference for gauging seismic design strength requirements for other components that are intended to remain elastic. In particular, the seismic demand on the column splice should be limited to a reasonable percentage of its nominal capacity compared to the deformation demand on the beam-to-column connection for any given ground motion intensity. This comparative approach provides a solid basis for developing a "capacity design" method for column splices in a



(a) Location of the column splice in Primary Case (PC)



(b) Location of the column splice in Secondary Case (SC)

Fig. 4. Locations of column splices evaluated.

Table 4. Ground Motions Used in the Study					
SAC Name	Record	Earthquake Magnitude	Distance (km)	Scale Factor	PGA (g)
LA21	1995 Kobe	6.9	3.4	1.15	1.28
LA22	1995 Kobe	6.9	3.4	1.15	0.92
LA23	1989 Loma Prieta	7	3.5	0.82	0.42
LA24	1989 Loma Prieta	7	3.5	0.82	0.49
LA25	1994 Northridge	6.7	7.5	1.29	0.87
LA26	1994 Northridge	6.7	7.5	1.29	0.94
LA27	1994 Northridge	6.7	6.4	1.61	0.93
LA28	1994 Northridge	6.7	6.4	1.61	1.33
LA29	1974 Tabas	7.4	1.2	1.08	0.81
LA30	1974 Tabas	7.4	1.2	1.08	0.99
LA31	Elysian Park (simulated)	7.1	17.5	1.43	1.30
LA32	Elysian Park (simulated)	7.1	17.5	1.43	1.19
LA33	Elysian Park (simulated)	7.1	10.7	0.97	0.78
LA34	Elysian Park (simulated)	7.1	10.7	0.97	0.68
LA35	Elysian Park (simulated)	7.1	11.2	1.1	0.99
LA36	Elysian Park (simulated)	7.1	11.2	1.1	1.10
LA37	Palos Verdes (simulated)	7.1	1.5	0.9	0.71
LA38	Palos Verdes (simulated)	7.1	1.5	0.9	0.78
LA39	Palos Verdes (simulated)	7.1	1.5	0.88	0.50
LA40	Palos Verdes (simulated)	7.1	1.5	0.88	0.63

special steel moment frame, in which the only designated energy dissipation portion is at the end of the beam, and all other portions (including the column splice, in the frame) are designed to remain essentially elastic with a reasonable margin of safety. This concept would still be expected to apply if inelastic deformation of the column panel zone were anticipated by the designer.

SEISMIC RESPONSE OF FRAME SYSTEMS AND PERFORMANCE CRITERIA

Inelastic dynamic analyses were conducted to study seismic column splice demands of three frames. The frames were subjected to 20 ground motion accelerations with spectral ordinates defined as having 2% probability of exceedance in 50 years. The frames were modeled as beam and column elements with potential plastic hinges at their ends. The interaction between the axial force and bending moment was considered in columns. A 5% strain-hardening ratio was assumed in the plastic hinges. P-Δ effects were always included in the time-history analyses. Modal analyses were conducted prior to time-history evaluations and indicated that the fundamental period of vibration of the 4-, 9- and 20-story frames is 0.80 s, 1.60 s and 2.40 s, respectively.

The dynamic response of the frames to the selected 20 ground motions showed dramatic variations throughout the suite of time histories, ranging from elastic behavior to near collapse. Thus, the suite of ground motions provided a wide range of demand on the seismic column splices. Two response indices—interstory drift ratio and plastic hinge rotation at the beam end—were chosen to represent the system performance. Based on these system response indices, the seismic performance of the three frames was divided into general categories based on the severity of observed interstory drift ratio and plastic hinge rotation at the beam end. Severity was judged qualitatively based on the amount of interstory drift ratio and plastic hinge rotation demands.

Seismic Response Category and Ground Motion Group

Seismic response of frame systems, in terms of peak story drift and plastic hinge rotation at beam ends, was used to categorize the intensities of ground motions for each individual frames. The response of each frame demonstrated dramatic differences among the 20 ground motions and was divided into groups based on peak story drift ratio and plastic hinge rotation.

Table 5 presents 20 time histories divided into three ground motion groups for each of the frames, named GMG1, GMG2 and GMG3, respectively, in order of increasing peak story drifts and plastic hinge rotation at beams (i.e., from mild to very severe response). This is a shifting of the time histories that make up each group, depending on which frame is being considered. Figures 6, 7 and 8 present the system responses in terms of (1) peak story drift ratio and (2) peak plastic hinge rotation at beam ends in 4-, 9- and 20-story frames, respectively. The data show that the peak drift and plastic hinge rotation response of 4-, 9- and 20-story frames vary dramatically over the 20 ground motions, and have been divided into three qualitative groupings corresponding to GMG1, GMG2 and GMG3, as just described. It appears that the set of 20 ground motions used in this study serves the purpose of evaluating a range of seismic demands on column splices in a reasonable manner.

In order to evaluate seismic demand of column splices, the system responses of the 4-, 9- and 20-story frames to the 20 ground motions, represented by story drifts and plastic hinge rotations, are divided approximately into three seismic response categories (SRC) as described in Table 6. The SRC is defined as follows:

1. SRC I, Functional to Moderate Structural Damage: the structure is likely to be functional with limited inelastic deformation in a small number of beams having less than 0.02 rad plastic hinge rotation in any beam.

2. SRC II, Near Life-Safety: the structure is expected to sustain moderate to heavy structural damage to

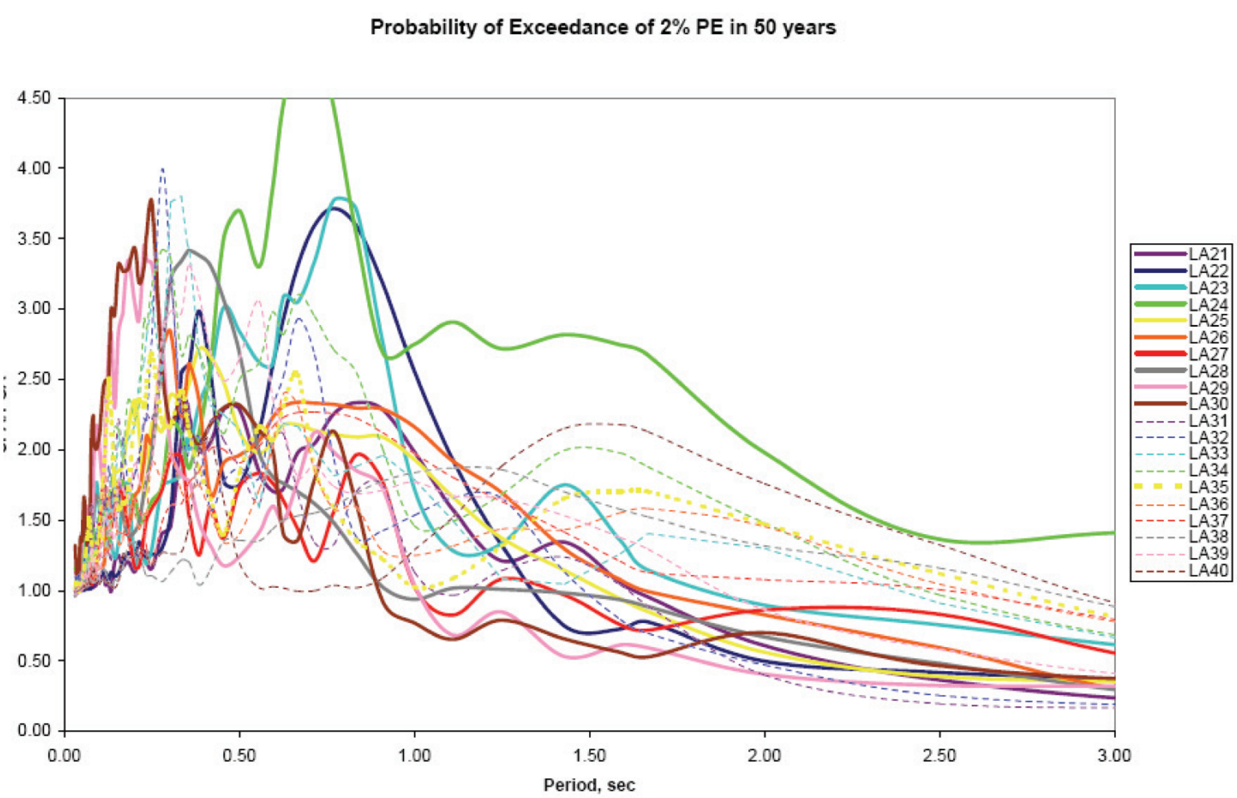


Fig. 5. Response spectra of the ground motions used in the seismic analyses.

Table 5. Ground Motion Group (GMG)			
Frame	GMG 1	GMG 2	GMG 3
4-story	LA23, LA24, LA27, LA28, LA29, LA30, LA31, LA33, LA34, LA35, LA39, LA40	LA21, LA22, LA25, LA26, LA32, LA36, LA37, LA38	Not Applicable
9-story	LA23, LA29, LA30, LA39	LA21, LA24, LA25, LA26, LA28, LA32, LA34, LA37	LA22, LA27, LA31, LA33, LA35, LA36, LA38, LA40
20-story	LA23, LA29, LA31, LA39	LA21, LA22, LA25, LA27, LA28, LA30, LA32, LA33, LA34, LA37, LA40	LA24, LA26, LA35, LA36, LA38

Table 6. System Response Category (SRC) of the Steel Moment Frames			
SRC	GMG	Story Drift Ratio (SDR)	Plastic Hinge Rotation (PHR)
I	1	0.01 to 0.02	0.01 to 0.02
II	2	0.02 to 0.04	0.02 to 0.04
III	3	0.04 to 0.05	0.04 to 0.05

its connections with many beam ends having plastic hinge rotations on the order of 0.02 to 0.04 rad.

3. SRC III, Life-Safety to Near Collapse: the structure has extensive and widely spread plastic hinge rotations on the order of 0.04 to 0.05 rad in many beams.

It is noted that the four-story frame suffers consistently lower system response than that in other two taller frames, and does not experience SRC III response.

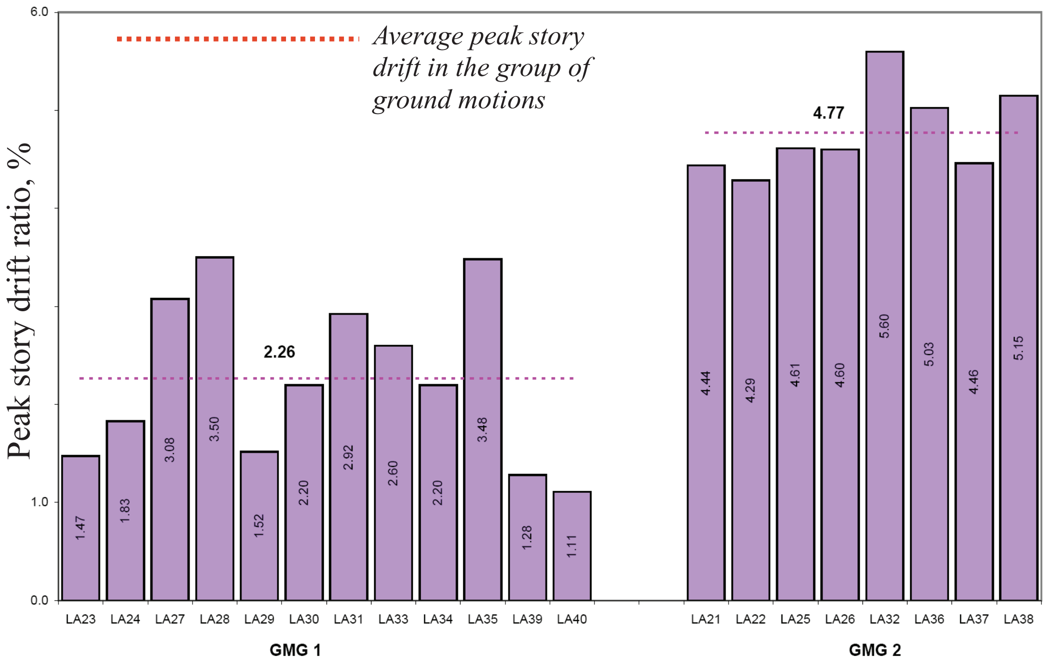
Correlations between Column Splice Demand and System Response

The response indices, story drift ratio (SDR) and plastic hinge rotation (PHR), together with more detailed information about structural response of the three frames subject to the 20 ground motions (Shen and Sabol, 2008), have demonstrated that it is possible for the seismic demand on the column splice to be directly related to the response of the system, instead of ground motions themselves, because of strong correlation between the column splice demand and system response. This suggests the following observations:

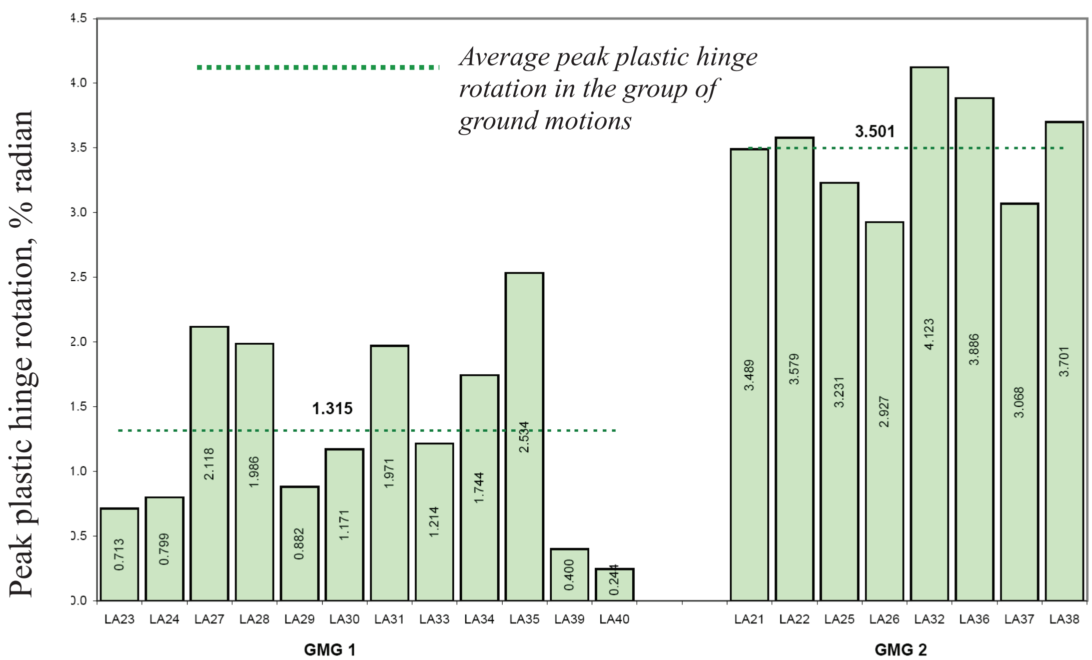
1. The ground motions that produce the maximum bending moments at column splices also result in maximum plastic hinge rotations at beam ends, indicating a close correlation between the bending moment in the column splice and the level of inelastic deformation in the structural system as a whole.
2. Extensive beam plastic hinge rotations in the frames subject to GMG 3 ground motions cause some columns in the 9- and 20-story frames to bend in single curvature and, in some extreme events, force some exterior columns to form plastic hinges at their ends under combined tensile axial force and bending moment. This behavior leads to significant bending moments at column splices and high tensile forces in interior columns. The frames in these events would be near their collapse thresholds with beam plastic hinge ration approaching or exceeding 0.06 rad.
3. The formation of plastic hinges in exterior columns due to high beam-end plastic hinge rotations under GMG 3 is responsible for unusually large P_s/P_{yt} ratios in some interior columns.

Maximum plastic hinge rotations on the order of 0.06 rad are expected to have resulted in extensive damage, evidenced by the fact that the ultimate plastic hinge rotation capacity of special moment frame beam-to-column connections under seismic loads are expected to be in the range of 0.04 to 0.06 rad. An evaluation of seismic design requirements for column splices must answer the question of whether or not the splice is adequate to survive theses large demands in the beams with an adequate margin of safety.

When the frames are exposed to significant inelastic deformation demands, the analytic results suggest seismic demands on the splice that are different from those conventionally assumed to occur based on an elastic analysis. For

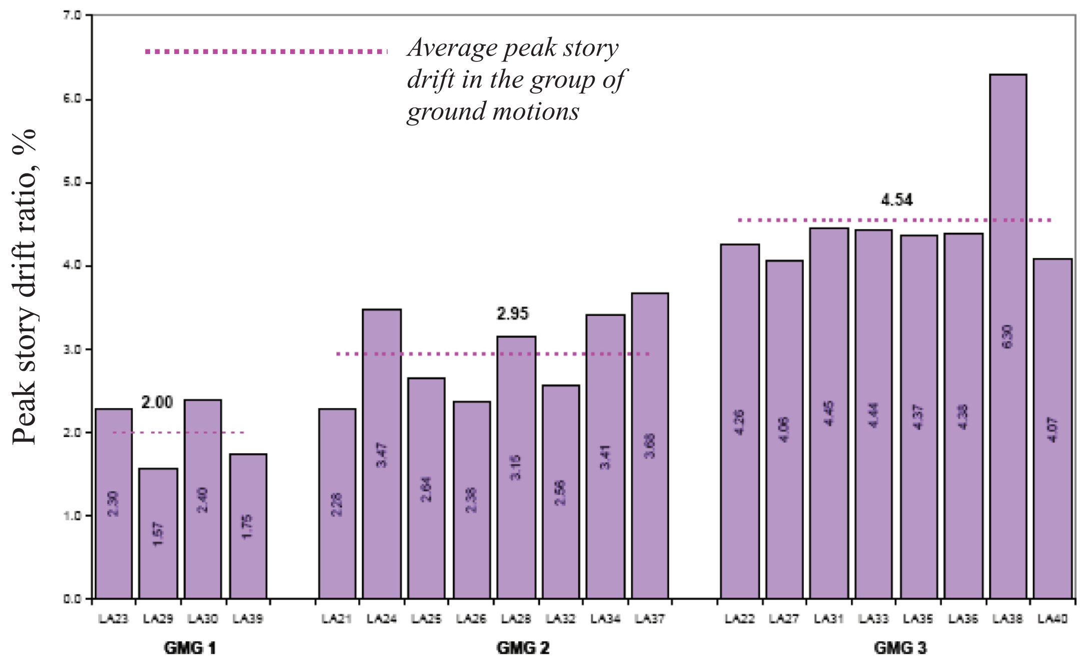


(a)Peak story drift ratio

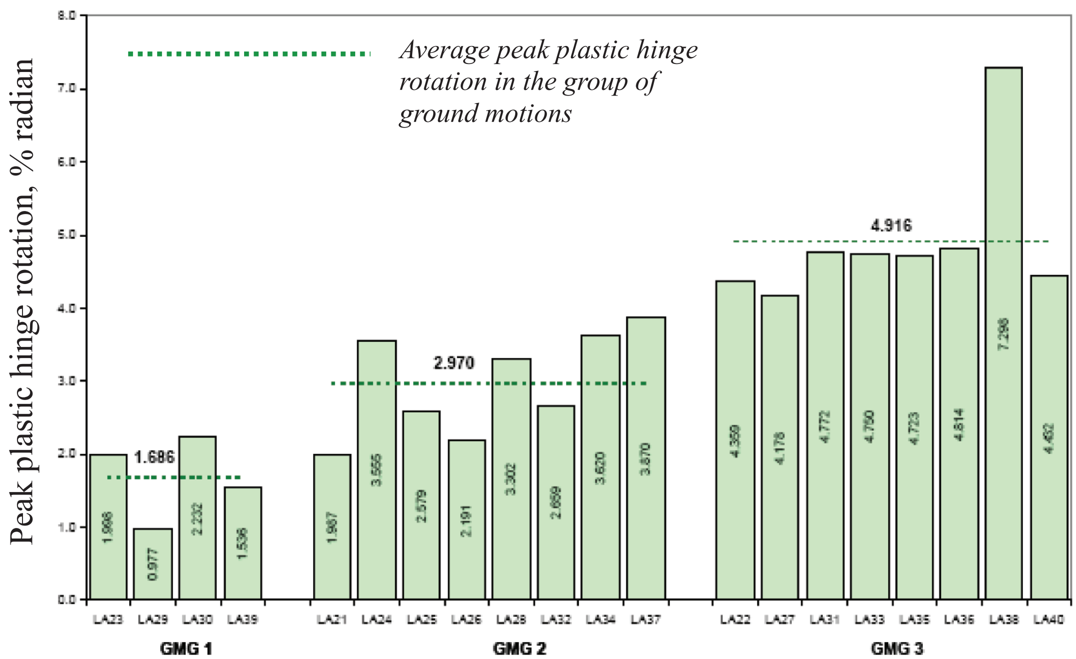


(b)Peak plastic hinge rotation at beam ends

Fig. 6. System response of the four-story frame subjected to the 20 ground motions.

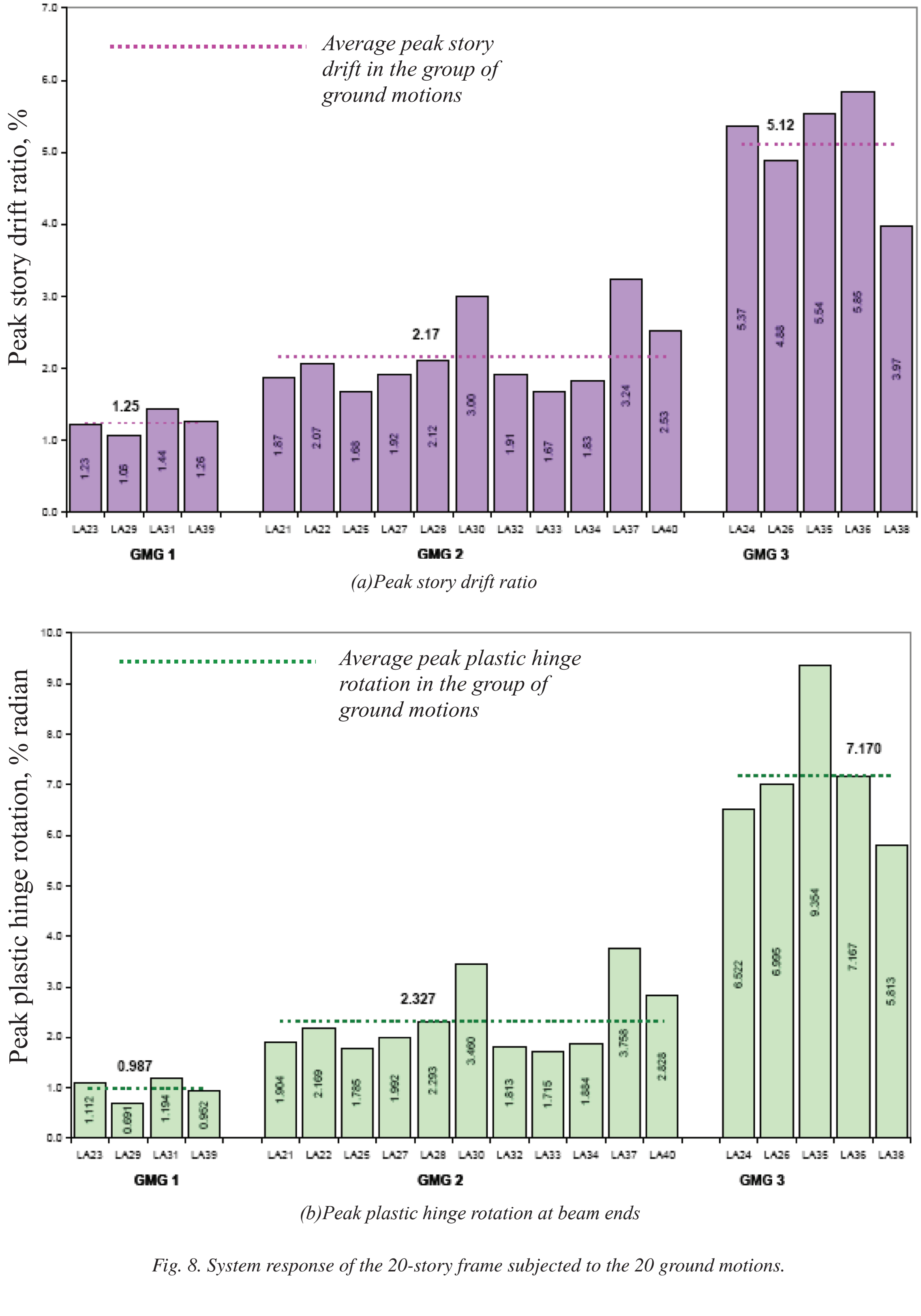


(a)Peak story drift ratio



(b)Peak plastic hinge rotation at beam ends

Fig. 7. System response of the nine-story frame subjected to the 20 ground motions.



(a)Peak story drift ratio

(b)Peak plastic hinge rotation at beam ends

Fig. 8. System response of the 20-story frame subjected to the 20 ground motions.

Table 7. $\frac{M_s^{PC}}{M_s^{SC}}$ Ratio Statistics						
Frame	GMG 1/SRC I		GMG 2/SRC II		GMG 3/SRC III	
	Mean μ	Standard Deviation σ	Mean μ	Standard Deviation σ	Mean μ	Standard Deviation σ
4-story	0.57	0.13	0.92	0.07	Not Applicable	
9-story	0.68	0.03	0.76	0.07	0.79	0.05
20-story	0.65	0.02	0.79	0.09	0.90	0.06

example, if we assume a point of inflection at mid-story and a straight line moment diagram with the maximum column moment at the beam centerline, it is expected that the column moment at the typical column splice location will be approximately 20% of that maximum moment. On the other hand, for GMG 2 and GMG 3, the average bending moment in the splice was found to be approximately 60% to more than 80% of the plastic moment capacity of the smaller column, as shown in Figures 9a, 10a and 11a.

SEISMIC DEMAND ON THE COLUMN SPLICES

Peak Bending Moment at the Column Splice, M_s

The peak bending moment in all column splices in a given frame, M_s, normalized by M_{pt}, the plastic moment of the smaller column on the top of splice, is summarized in Figures 9a, 10a and 11a for the 4-, 9- and 20-story frames, respectively, subjected to the suite of 20 ground motion time histories. Two different structural models, primary case (PC) and secondary case (SC), were analyzed. The only difference between the two models was the column splice location. In the PC model, the column splices were placed 4 ft above the floor slab. In the SC model, the column splices were assumed 4 ft above the beam center line, which serves as a reference to compare with the PC model in order to discuss the influence of the column splice location on column splice demand.

The difference in the bending moment at the column splice, M_s, between two models is presented in Table 7 as the mean value and standard deviation of M_s^{PC}/M_s^{SC} ratio for the three frames studied, where M_s^{PC}, M_s^{SC} is the peak bending moment at any column splice in PC and SC models, respectively.

Influence of Column Splice Location on Moment Demand

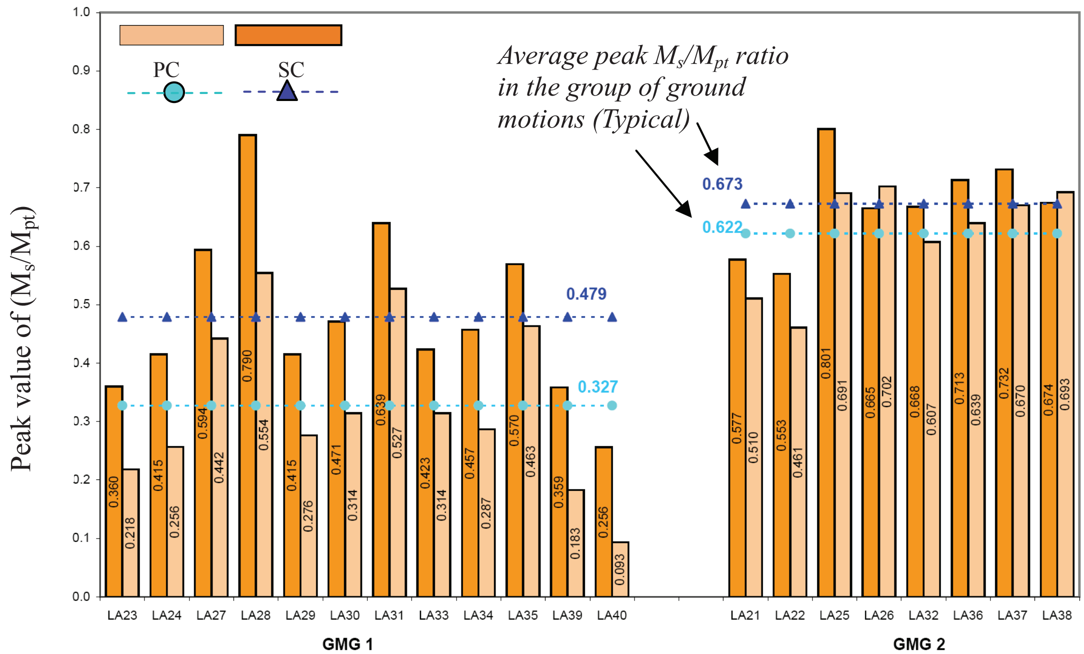
From Figures 9, 10, and 11 and Tables 7 and 8, one can observe the following:

1. The peak bending moment at column splices is generally lower from the PC model than the SC model. This is consistent with the general expectation that the column splice moment will generally be lower as the splice is moved away from the beam-column joint.
2. The difference in the peak bending moment between the PC and SC models appears to be significant when the frame system response is mild or moderately severe (i.e., response to GMG 1 and GMG 2); however, such differences become less significant when the structure experiences large inelastic deformation (i.e., responses to GMG 3). As shown in Table 7, the difference in moment demand at column splices between the two models was about 30% to 40% when seismic response of the frames was mild (in GMG 1/SRC I case) and about 10% to 20% when seismic response of the frame was moderately severe (in GMG 2/SRC II case) in all three frames. When seismic response of the frames was very severe (in GMG 3/SRC III case), the difference in moment demand at column splices between two models is about 10% in the 20-story frames, and about 20% in the 9-story frame. This suggests that the moment gradient is less steep than it is for the smaller seismic demand. The less steep moment gradient would be consistent with columns bent in single curvature.

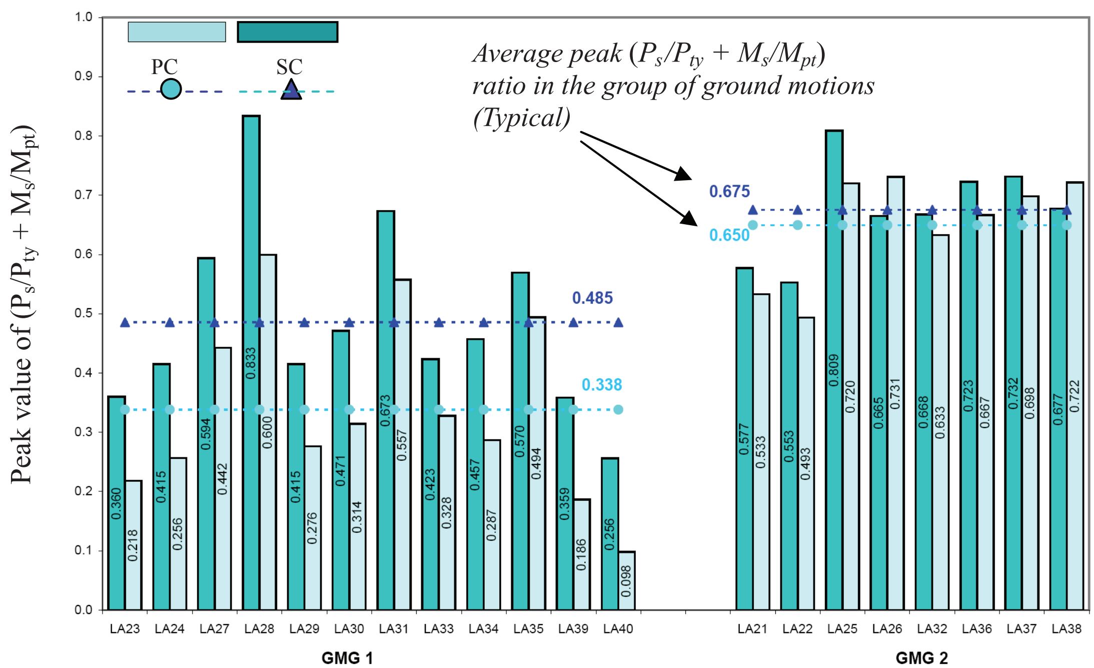
Bending Moment Demand on Column Splice with Respect to Seismic Response of the Frame

The following discussions focuses on the general trends of column splice moment demand with respect to the severity of structural response based on the PC model since the results from both PC and SC models have the similar trends. As shown in Table 9, the following points appear relevant:

1. When the frames had a mild (i.e., little or limited inelastic) response (GMG 1/SRC I response), the M_s/M_{pt} ratio at the column splice was consistently less than 0.40 in the four-story frame, and less than 0.35 in the 9- and 20-story frames.

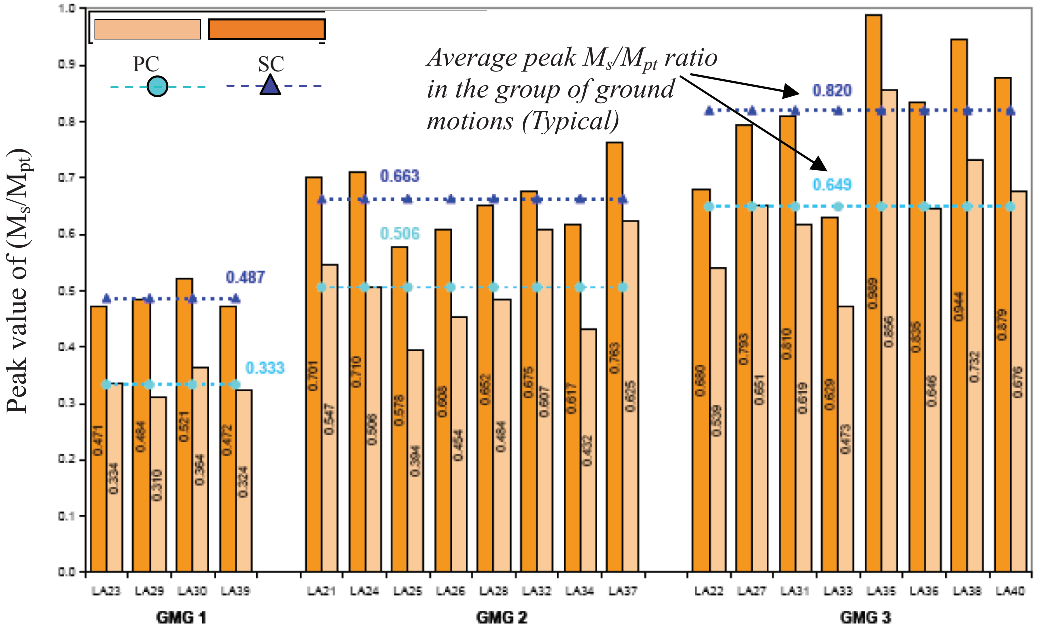


(a)Peak value of M_s/M_{pt}

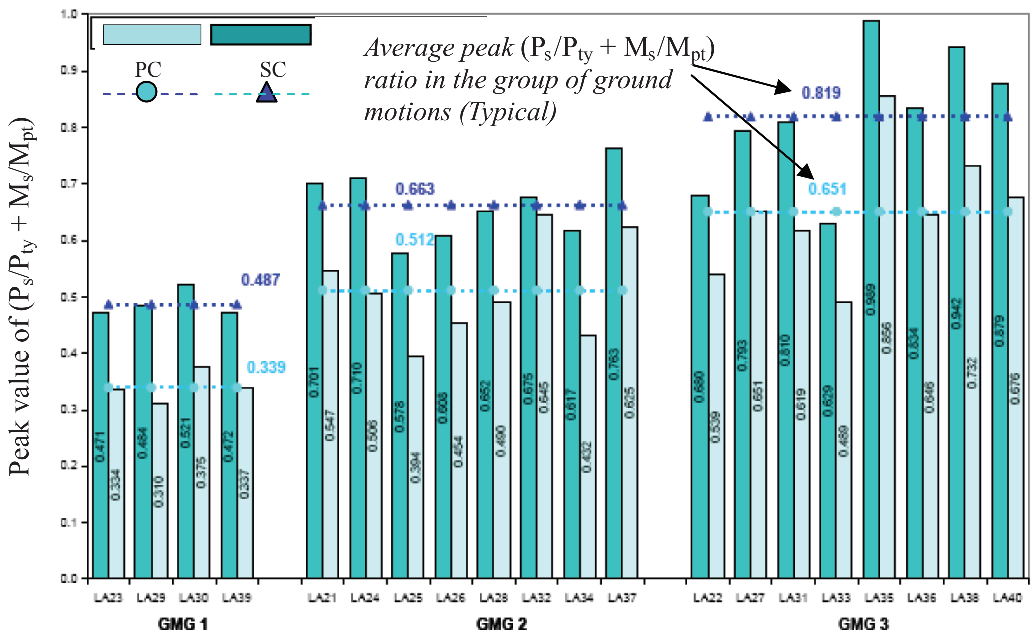


(b)Peak value of $(P_s/P_{ty} + M_s/M_{pt})$

Fig. 9. Column splice response of the four-story frame subjected to the 20 ground motions.

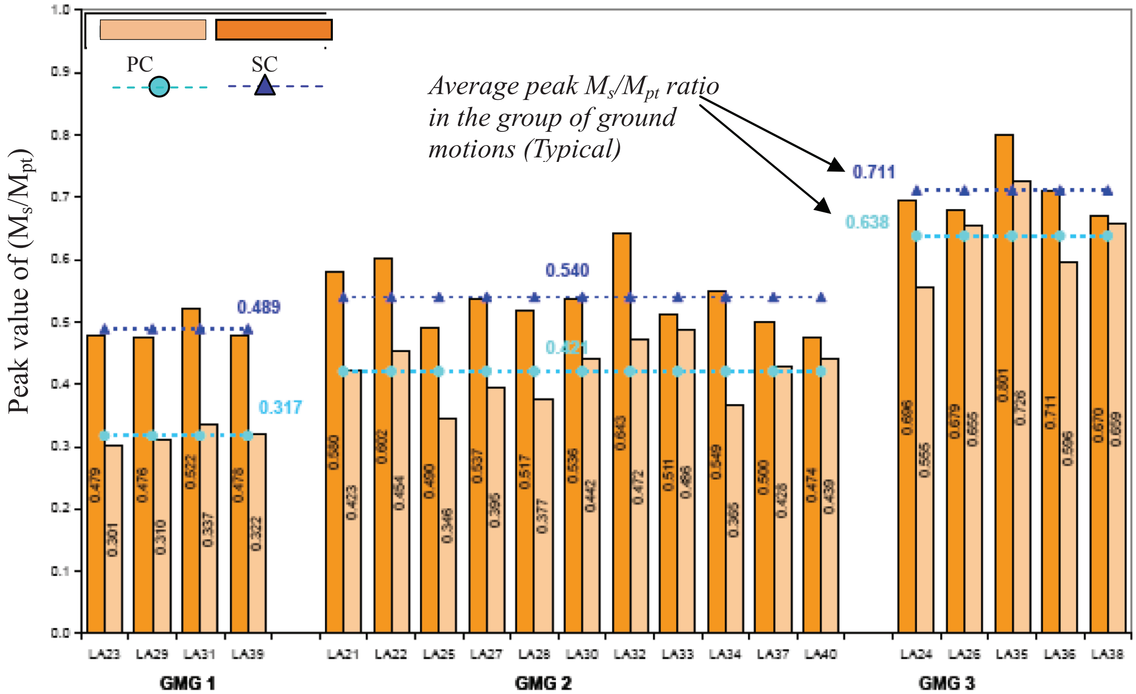


(a)Peak value of M_s/M_{pt}

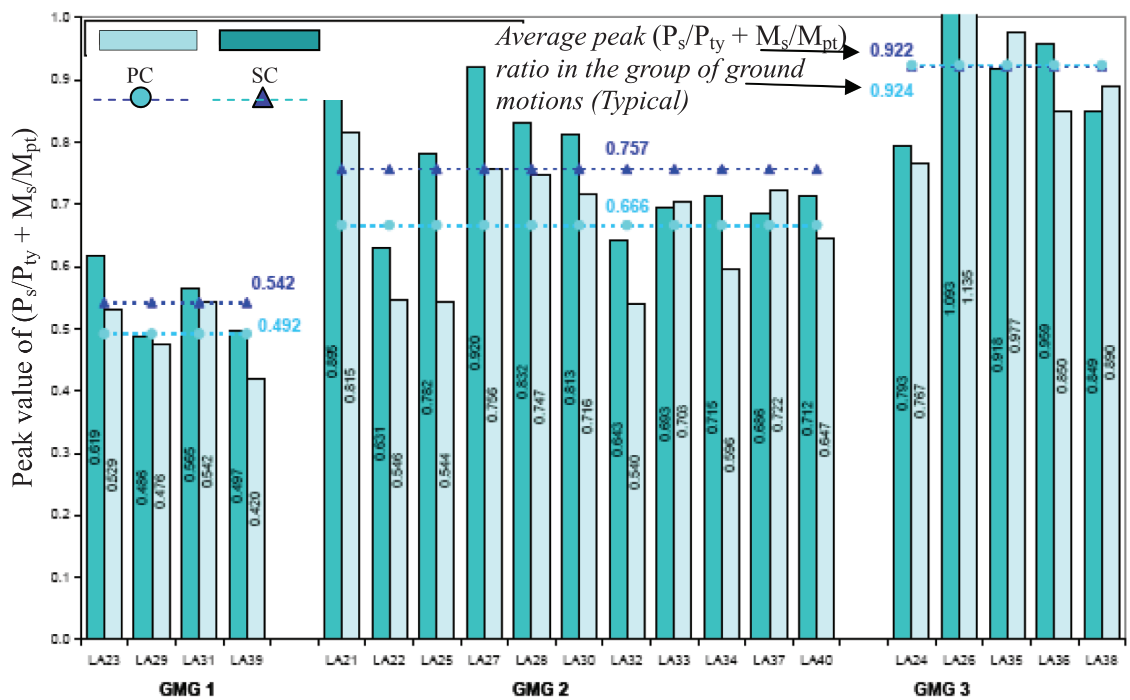


(b)Peak value of $(P_s/P_{ty} + M_s/M_{pt})$

Fig. 10. Column splice response of the nine-story frame subjected to the 20 ground motions.



(a)Peak value of M_s/M_{pt}



(b)Peak value of ($P_s/P_{ty} + M_s/M_{pt}$)

Fig. 11. Column splice response of the 20-story frame subjected to the 20 ground motions.

Table 8. $\frac{\left(P_S/P_{ty}+M_s/M_{pt}\right)^{PC}}{\left(P_S/P_{ty}+M_s/M_{pt}\right)^{SC}}$ Ratio Statistics						
Frame	GMG 1/SRC I		GMG 2/SRC II		GMG 3/SRC III	
	Mean μ	Standard Deviation σ	Mean μ	Standard Deviation σ	Mean μ	Standard Deviation σ
4-story	0.67	0.13	0.96	0.07	Not Applicable	
9-story	0.70	0.03	0.77	0.08	0.80	0.05
20-story	0.91	0.06	0.89	0.09	1.00	0.07

2. When the frames experience moderate to large amounts of inelastic deformation (GMG 2/SRC II response with beam end plastic hinge rotations between 0.02 and 0.04 rad), the M_s/M_{pt} ratio has a mean value of 0.622 with a standard deviation of 0.092 in the 4-story frames, a mean value of 0.506 with a standard deviation of 0.075 in the 9-story frame, and a mean value of 0.421 with a standard deviation of 0.043 in the 20-story frame.
3. When the frames experience extremely large inelastic deformation with the beam end plastic hinge rotations between 0.05 and 0.07 rad (GMG 3/SRC III response), the M_s/M_{pt} ratio has a mean value of 0.649 with a standard deviation of 0.100 in the 9-story frame and a mean value of 0.638 with a standard deviation of 0.059 in the 20-story frame.

Peak Combination of Bending Moment and Axial Tensile Force in the Column Splice

Axial tensile forces can result in significant demands on the column splice, particularly in a taller frame, such as the 20-story frame examined in this study. Figures 9b, 10b and 11b summarize the peak combination of normalized bending moment and axial tensile force in column splices in the 4-, 9- and 20-story frames, ($P_s/P_{ty} + M_s/M_{pt}$), where P_s is the tensile force at the splice, and P_{ty} (= F_yA_g) is the nominal tensile strength of the smaller column. The results from two structural models, PC and SC, are plotted in the same figures for comparison.

Influence of Column Splice Location on the Combined Moment and Axial Force Demand

Because the axial force demand at the column splice is independent of splice location, the total seismic demand on a column splice, in terms of combined moment and axial force, appears less affected by the splice location when axial force demand is significant. From Figures 9b, 10b and 11b and Table 8, one can make the following observations:

1. Peak combined moment and axial force demand at column splices is consistently lower in the PC model than in the SC model for all three frames when the frame system response was mild or moderately severe (in GMG 1/SRC I and GMG 2/SRC II cases). This is consistent with the observation that overall peak demand is highly correlated with peak flexural demand and that splice location does not affect peak tensile demand for less significant ground motions in short- or medium-height frames.
2. When the seismic response of the frames was very severe (in GMG 3/SRC III case), the difference in the combined moment and axial force demand due to different splice locations became less insignificant. In particular, the peak combined moment and axial force demand in the 20-story frame appears to be independent of the column splice location. It was noted, for example, that the demands at the column splice are higher in the PC model than those in the SC models for some ground motions in GMG 3 group, but for other ground motions in the group the opposite was true. This suggests that the maximum combined demand was significantly more sensitive to axial forces in the taller frames.

Combined Moment and Axial Force Demand on Column Splice with Respect to Seismic Response of the Frame

Based on a review of the PC model simulation data, the following general trends were observed for combined moment and axial force demand at a column splice with respect to the severity of structural response:

1. Tensile axial forces in columns in taller frames may be significant for frames that have experienced moderate or extensive inelastic deformation.
2. A more significant impact of the tensile axial force on the column splice is observed in a taller frame even under less severe ground motions. In particular, the peak

Table 9. Summary of Seismic Response (of PC Model)					
Ground Motion Group (GMG)	System Response Category (SRC)	Building Type	Peak System Response (PHR) $\mu \pm \sigma$	Peak Demand on Column Splices	
				M_s/M_{pt} $\mu \pm \sigma$	$P_s/P_{ty} + M_s/M_{pt}$ $\mu \pm \sigma$
1	I	4-story	0.013 ± 0.007	0.327 ± 0.135	0.338 ± 0.141
		9-story	0.017 ± 0.008	0.333 ± 0.020	0.339 ± 0.020
		20-story	0.010 ± 0.002	0.317 ± 0.013	0.492 ± 0.048
2	II	4-story	0.035 ± 0.004	0.622 ± 0.092	0.650 ± 0.085
		9-story	0.030 ± 0.007	0.506 ± 0.075	0.512 ± 0.080
		20-story	0.023 ± 0.007	0.421 ± 0.043	0.666 ± 0.089
3	III	4-story	Not Applicable	Not Applicable	
		9-story	0.050 ± 0.009	0.649 ± 0.100	0.651 ± 0.090
		20-story	0.070 ± 0.012	0.638 ± 0.059	0.924 ± 0.125

combination of normalized bending moment and axial tensile force of the column splice in the 20-story frame has reached a mean ratio of 0.666 with a standard deviation of 0.089 when the frame has 0.02 to 0.03 rad plastic hinge rotation subjected to GMG 2 ground motions. This ratio reaches a mean value of 0.924 with a standard deviation of 0.125 when the frame has around 0.07 rad plastic hinge rotations.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

For the 4-, 9- and 20-story frames reviewed in this study, column splice demand based on a structural model assuming the column splice located 4 ft above the floor (i.e., the PC model), the following conclusions can be drawn:

1. The response of all three types of frames to the selected 20 ground motions, all defined as having a 2% probability of exceedance in 50 years, may be qualitatively divided into three different seismic response categories (SRC) consisting of little to moderate structural damage (SRC I system response), moderate to severe structural damage (SRC II system response), and near collapse (SRC III system response), respectively. In other words, the frames, representing low to moderately tall moment frames, show a wide range of seismic response to the 20 ground motions, as can be seen in Table 9.
2. The seismic demand on the column splice appears to be closely related to primary system response indices such as the magnitude of plastic hinge rotations at the beam end. These facts enable us to evaluate the seismic demand on the column splice based on the response of the frame to the selected ground motions, rather than solely on the ground motions themselves.
3. When the seismic response of the frame is low to moderate, a comparison between different column splice locations (e.g., the PC and SC models) suggests that the bending moment demand at a splice closer to the column mid-height (i.e., the PC model) is consistently lower than when the splice is taken at the beam-column joint centerline.
4. When frames experience heavy inelastic deformation (e.g., the SRC III case), the influence of the column splice location on observed splice demand becomes less significant. In particular, when axial force is a significant contributor to overall splice demand, the seismic demand at the column splice appears to be independent of splice location.
5. The peak bending moment at a column splice may reach 60% of the flexural strength of the smaller column when the maximum plastic hinge rotations are less than 0.04 rad (i.e., SRC I and II response) and up to 70% to 80% of flexural strength of the smaller column when the maximum plastic hinge rotations are between 0.05 and 0.07 rad (i.e., SRC III system response).
6. The significant impact of applied tensile axial forces on the column splice is observed in taller frames even under less severe ground motions (e.g., GMG 2). Some column splices appear to experience demand-to-capacity ratios (D-C) considering peak combined bending moment and tensile demand of up to 0.8 when the maximum plastic hinge rotation is as low as 0.02 rad and between 0.9 and 1.0 when the maximum plastic hinge rotation is on the order of 0.07 rad.

7. Given the many uncertainties inherent in these types of analyses, it would be reasonable to anticipate at least SRC II system response when a frame is subjected to the 2% probability of exceedance design earthquake. SRC III system response is certainly possible for some types of ground motions and may be relatively more frequent for taller frames. Demand on the column splice can be on the order of the smaller column's strength when the critical beam-to-column connection reaches its expected maximum deformation capacity.

Based on this analytical study, the following recommendations are suggested:

1. Until additional research considering combined cyclic flexural and tensile actions reduces the uncertainty inherent in reliably estimating the capacity of a welded column splice constructed using partial-joint-penetration groove welds, it is recommended that a significant margin of safety be provided for column splices in seismic load-resisting structures.
2. For special moment frames in moderately tall structures (e.g., those taller than approximately nine stories, where the effects of tensile axial loads on column splices may be significant), current requirements mandating use of complete-joint-penetration groove welds in welded column splices appears reasonable.
3. For special moment frames in shorter structures (e.g., those less than or equal to approximately nine stories), current requirements mandating use of complete-joint-penetration groove welds in welded column splices appear conservative. Welded splices using partial-joint-penetration groove welds (or the equivalent bolted splice) designed to develop at least $0.8M_p$ of the smaller column appear to provide a reasonable margin of safety and could be permitted.
4. Based on the seismic demands on the column splices determined from this study, it is recommended that an experimental and analytical study be undertaken to investigate the performance of column splices using partial-joint-penetration groove welds (or equivalent bolted splices) under combined cyclic flexural and tensile axial force demands. This additional research could also investigate the reliability of the proposed height limitations proposed (e.g., above and below nine stories) using methodologies such as those outlined in ATC-63 (FEMA P695 [FEMA, 2006]).

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