SEISMIC DEMAND ON COLUMN SPLICES IN STEEL MOMENT FRAMES

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ABSTRACT :

This study addresses concerns related to safety and economy of seismic column splices in steel moment-resisting frames, leading to a balanced seismic design of column splices. A comprehensive analytic investigation consisted of the seismic response analysis of three moment-resisting frames with 4, 9, and 20 stories, respectively, subjected to an ensemble of 20 strong ground motions. The outcomes of the study include comprehensive seismic demand on the column splice and recommended guidelines for a safe and economical design of column splices. The study concludes that the strength demand on the column splice would be on the same order as that of the smaller column when the critical beam-to-column connection reaches its expected maximum deformation capacity, and that it does not appear unreasonable for the current seismic design provisions on the column splice in special moment frames to require the column splice to develop the strength of the smaller column.

KEYWORDS: Column splice, steel moment frame, seismic design

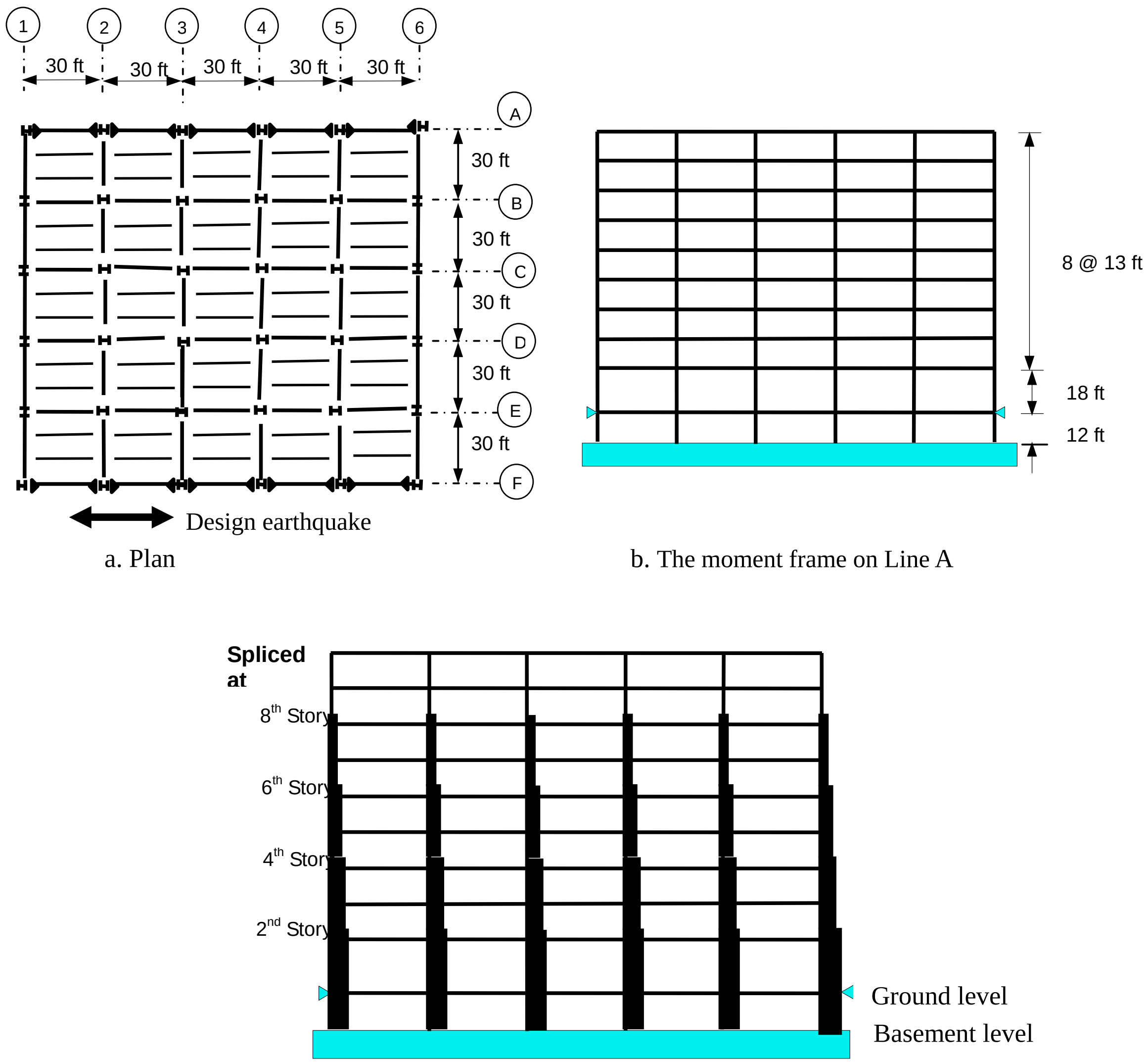
1. INTRODUCTION

Special moment frames (SMRF) have been one of the most frequently used seismic force resisting systems in steel building structures throughout the United States for nearly a half century. During the 1994 Northridge earthquake, some steel moment frames with welded moment connections suffered damage at or near their beam-to-column joints. Since then, the structural engineering and steel construction communities undertook an extensive research effort, centering on the beam-to-column connection, to investigate the damage, and ultimately improve seismic design, construction, inspection, evaluation and retrofit of the steel moment frames. This resulted in much improved understanding of seismic demand and capacity of beam-to-column connections in steel moment frames. In addition to enhanced requirements for connection design, the current seismic design provisions (AISC 341) also require that column splices in moment frames, when not made using complete joint penetration (CJP) welds, shall be designed to develop the flexural strength of the smaller connected column and the shear demand associated with flexural hinging at the top and bottom of a column at a given story assuming a point of inflection at mid-height. The following two issues appear to play a role in the seismic design practice of column splices: (1) Welded connections of heavy steel sections turned out to be much more susceptible to brittle fracture than was commonly acknowledged before the 1994 Northridge Earthquake. It was assumed that partial joint penetration (PJP) welds in some configurations were more susceptible than CJP, due to high levels of stress concentration. Thus, it appeared reasonable to require CJP welds in lieu of PJP welds in column splices because of the higher levels of inelastic performance anticipated when the ductility of beam-to-column connection was improved; and (2) The demand on the 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 adjacent to the beam-column joint when subjected to the (elastic) equivalent lateral design seismic forces. Based on this reasoning, in order for the column splice to develop a plastic hinge, the beam-to-column connection needs to go through a significant amount of plastic rotation. It was assumed that the beam-to-column connection would reach its critical limit state before the column splice does. The question arises whether the seismic design provisions regarding column splices in the current AISC Seismic Provisions can be justified to require a seemingly more conservative design approach for column splices than the past. The revised column splice requirements can lead to significantly increased cost due to heavy welds and erection aids necessary to stabilize the column prior to welding. Given the limited detailed research on this topic, it appeared prudent to conduct a systematic seismic investigation on column splices to address the question of whether the newly added seismic provision requiring CJP groove welds at column splices is justified or unnecessarily conservative. This paper presents the seismic demand on the column splices with respect to that on the frame system as whole and beam-to-column connection in particular so that the influences of uncertainty, such as types of ground motions and properties of the structural systems, on the results might be minimized.

2. STRUCTURES AND GROUND MOTIONS

2.1. 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, were designed based on the seismic design requirements in ASCE 7-05 (ASCE 7) and AISC 341-05 (ASIC 341). A typical plan and elevation of the 9-story frame are given in the Figure 1, and the member sizes are included in Table 1. Braced frames (not shown in the figure) are used in the direction perpendicular to the moment frames. The 4- and 20-story frames are conceptually similar.



a. Plan

b. The moment frame on Line A

c. Splice locations in the 9-story frame

Figure 1. Plan and Elevation of the 9-story frame

Table 1. Member sizes of the 9-Story Frame

Level	Exterior Column	Interior Column	Beam
9	W14×257	W14×311	W24x55
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

Since the computer model used in the analysis employed a centerline representation of the framing, the design assumed that the column splices were located 4 feet above the center line of the girder. This is believed to be somewhat conservative because it locates the splice closer to the beam-column connection than is permitted by the AISC Seismic Provisions. (i.e. at least four feet from the beam-column connection per Section 9.9). The four foot 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 low as long as the building remains in the elastic range. The column splices are located at every other floor of the nine-story frame. The structural system for each building consists of steel perimeter moment resisting frames and interior simply-connected framing for gravity, i.e. lateral loads are carried by perimeter frames and interior frames are not explicitly designed to resist seismic loads in the direction of the earthquake.

The footprint of each building is symmetrical. The 4-story building has plan dimensions of 120 ft x 180 ft with a typical story height of 13 ft and consists of four-bay and six-bay frames in two orthogonal directions, respectively, spaced at 30 ft. The columns are assumed to be fixed at the ground level. The 9-story building has plan dimensions of 150 ft x 150 ft and consists of five-bay frames in both orthogonal directions spaced at 30 ft. The building has a basement level (B1 in Figure 1b). The typical story height is 13 ft except at the ground and basement levels where it is 18 ft and 12 ft, respectively (Figure 1b). The 20-story building has plan dimensions of 100 ft x 120 ft and consists of five-bay and six-bay frames in orthogonal directions, respectively, spaced at 20 ft. The building has two basements levels. The typical story height is 13 ft except at the ground and basement levels where it is 18 ft and 12 ft, respectively. The columns are assumed to be pinned at the basement level for the 9- and 20-story buildings, respectively, although they run continuously through the ground level framing. For 9- and 20-story buildings, concrete foundation walls and surrounding soil are assumed to prevent any horizontal displacement of the structure at the ground level.

The buildings were designed for a site in downtown Los Angeles where S_S is 200%g and S_1 is 100%g. The perimeter frames of the buildings in the direction of the design earthquake were designed as special moment frames and a response reduction factor of R = 8 was used. The ASCE 7 (ASCE 7) base shears corresponding to the 4-, 9-, and 20-story buildings were 1,438 kips, 1,946 kips, and 1,526 kips, respectively. The approximate period equation prescribed in the provisions was first used to check for strength before the drift requirements were evaluated. As expected, drift requirements governed the design for all the buildings.

2.2. Earthquake Ground Motions and Evaluation Method Employed in This Study

The fundamental philosophy in the seismic design of a special steel moment frame is to have ductile beam-to-column connections dissipating significant amounts of seismic energy through extensive inelastic deformation (damage) so that other structural parts of the frame, including column splices, remain functional and avoid collapse during the design earthquake event. Therefore, the seismic demand on the column splice ought to be compatible with that on the frame in general, and on the beam-to-column connection in particular. The compatibility 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 those on the frame as a whole regardless of types and intensities of the ground motions selected for the study. In this study, an ensemble of the ground motions was selected so that the seismic response of each of the three frames would be 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 with 2% Probability of Exceedance in 50 years (2%PE), titled LA21 to LA40, were used. Figure 2 presents the response spectra of these ground motions.

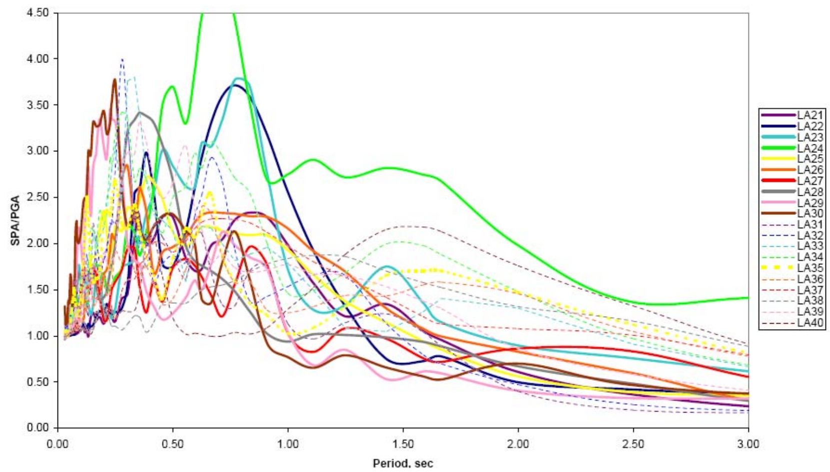


Figure 2. Response spectra of 2%PE in 50 years ground motions used in the seismic analyses

The seismic response evaluation of the column splice is based on two groups of response parameters: (1) the maximum plastic hinge rotations at each floor of the frame; and (2) 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}. 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 how severely the column splice responds the ground motion, leading to a rational design strength requirement for the column splice within the system, where all components are interrelated, and a desirable order in its possible failure chain is well defined. In particular, the beam-to-column connection in a special steel moment frame is the most critical element when subject to strong ground motions. The well documented seismic behavior of beam-column connections is a critical consideration in current seismic design provisions related to steel moment frames.

It is essential to relate seismic demand of the column splice to that of the beam-to-column connection. This comparative approach provides a solid basis for developing a "capacity design" method for column splices in a 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 elastic with a reasonable margin of safety. This concept would still be expected to apply in inelastic deformation of the column panel zone were anticipated by the designer.

The system response of each of three frames, 20-, 9- and 4-story frames to the twenty ground motions may be divided qualitatively into three groups for each one of three response categories as follows:

(1) Category I "Functional to minor structural damage" category, i.e., the structure is still functional with no or little inelastic deformation, i.e., less than 2% of plastic hinge rotation, when subject to Group 1 ground motions (LA23, LA24, LA29, LA39, and LA40 for the 4-story frame; LA23, LA29, LA30 and LA39 for the 9-story frame; and LA23, LA29, LA31 and LA39 for the 20-story frame)

(2) Category II "Near Life-safety" category, i.e., the structure suffers heavy structural damage to its connections with 2% to 4% plastic hinge rotation, when subject to Group 2 ground motions (LA26, LA27, LA28, LA30, LA31, LA33,

LA34, and LA35 for the 4-story frame; LA21, LA24, LA25. LA26, LA27, LA28, LA32, LA34, LA37, and LA40 for the 9-story frame; and LA21, LA22, LA25. LA26, LA27, LA28, LA30, LA32, LA33, LA34, LA37, and LA40 for the 20-story frame); and

(3) Category III "Life-safety" category, i.e., the structure has extensive and wide spread plastic hinge rotations in the order of 4% to 6% radians, when subject to Group 3 (LA21, LA22, LA25, LA32, LA36, LA37, and LA38 for the 4-story frame; LA22, LA31, LA33, LA35, LA36, and LA38 for the 9-story frame; and LA24, LA35, LA36, and LA38 for the 9-story).

3. SEISMIC RESPONSE

3.1. Peak Bending Moment in the Column Splice, M_s

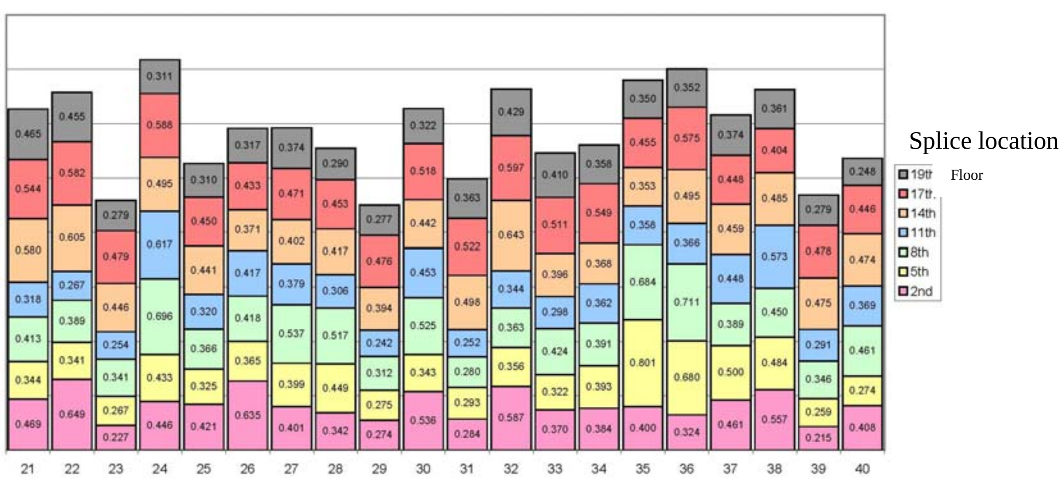
The fundamental period of vibration in 20-, 9-, and 4-story frames is 2.40 sec., 1.60 sec., and 0.80 sec., respectively. It has been observed that seismic response of the 20- and 9-story frames is influenced by higher modes. For all 20 time histories, the maximum bending moment in column splices at a given floor, M_s, normalized by M_{pt}, the plastic moment of the smaller column on the top of splice, is summarized in Figure 3 for the 9- and 20-story frames.

3.2. The Peak combination of bending moment and axial tensile force in the column splice

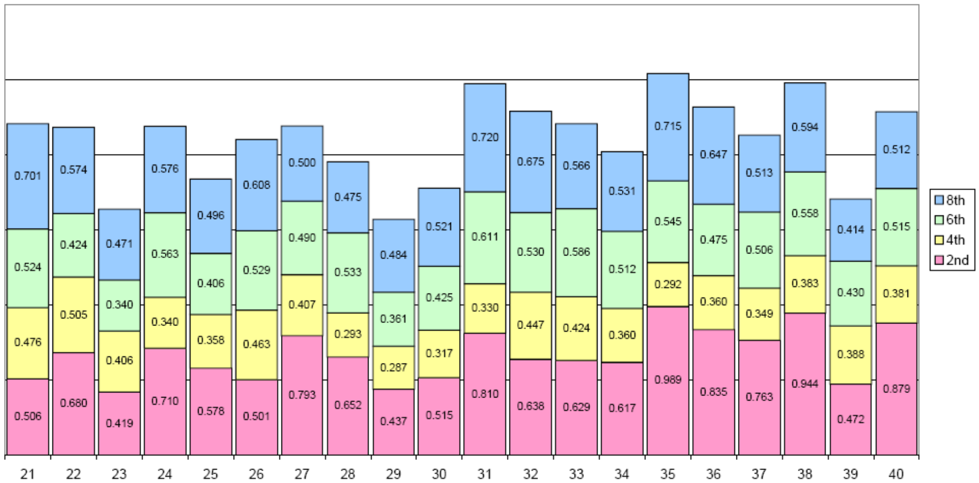
Figure 4 summarizes the peak combination of normalized bending moment and axial tensile force in column splices at a given floor, $(P_s/P_{ty} + M_s/M_{pt})$, where P_s is the tensile force at the splice, and P_{ty} is the tensile strength (= F_yA_g) of the smaller column in the 20-story frame.

3.3. The Plastic Rotations at the Beam End

Figure 5 summarizes the maximum, minimum, and average plastic hinge rotations at beam ends when the 20- and 9-story frames are subject to 20 ground motions.



(a) M_s/M_{pt} ratios at each column splice in the 20-story frame.



(b) M_s/M_{pt} ratios at each column splice in the 9-story frame.

Figure 3. M_s/M_{pt} ratios at each column splice in the 20-story and 9-story frames subjected to the 20 2%PE in 50 years ground motions, 21 (LA21) through 40 (LA40).

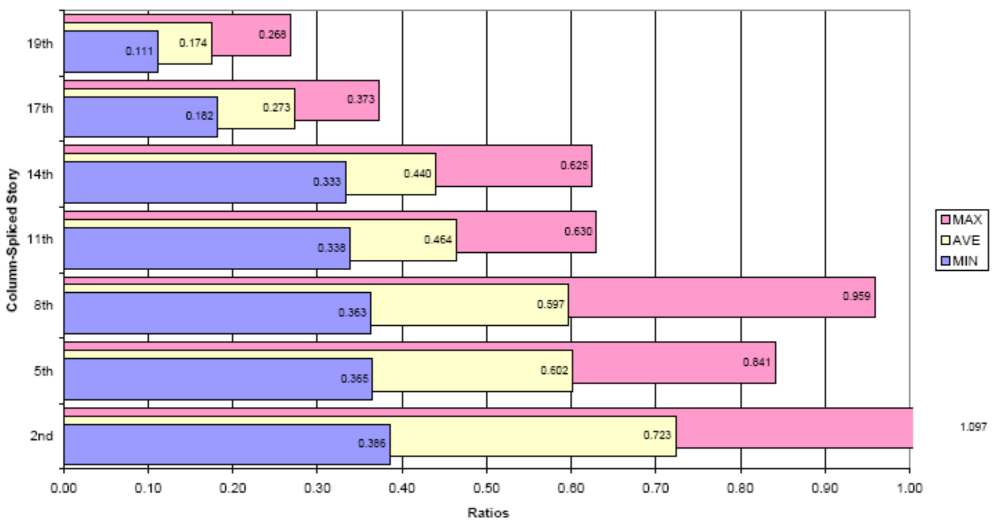


Figure 4. Maximum (MAX), Minimum (MIN) and Average (AVE) of (P_s/P_{ty} + M_s/M_{pt}) in the exterior column splices from the twenty 2% PE in 50 years ground motions.

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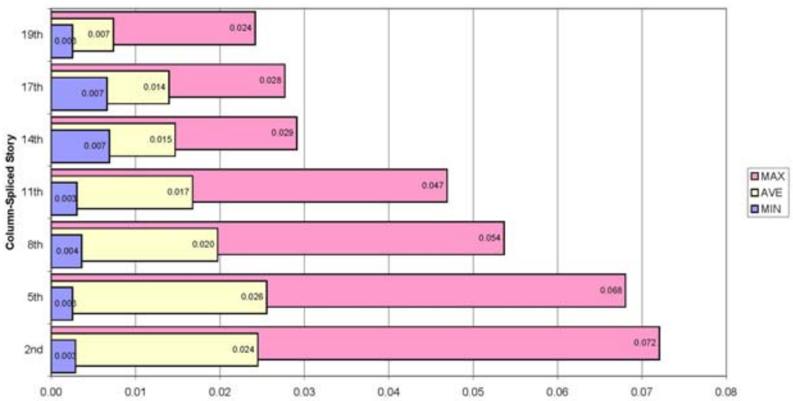
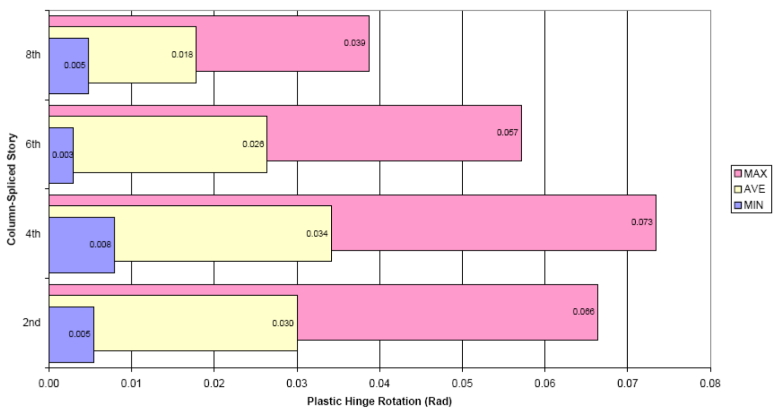


(a) 20-story frame



(b) 9-story frame

Figure 5. The maximum (MAX), minimum (MIN) and average (AVE) plastic hinge rotations (in radian) at beam ends subject to all 20 ground motions.

3. OBSERVATIONS AND CONCLUSIONS

Table 2 below summarizes the data from the response from the 20-, 9- and 4-story frames in terms of the column splice demand and system response. The following observations may be made:

(1) The response of all three types of frames to the selected 20 ground motions are divided qualitatively into three different categories with little to moderate structural damage (Category I system response), moderate to severe structural damage (Category II system response), and near collapse (Category III system response), respectively. In other words, three types of frames, representing low to moderately tall moment frames, show a wide range of response. This may or may not be surprising to engineers designing buildings based on current building code requirements. Furthermore, the seismic demand on the column splice is closely related to primary system response indices such as the magnitude of plastic hinge rotations of beams. These enable one 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.

(2) Single-curvature flexural deformation is possible in columns even under an expected inelastic system response (i.e. Category II), and is obvious in columns over many floors under large inelastic deformation in the Category III system response. In taller frames (9- and 20-story frames), extensive plastic hinge rotations in the frame with some Category II and many Category III system response are responsible for some columns bending in single curvature. This response can also force some exterior columns to form plastic hinges at their ends under combined axial force and bending moment, leading to significant bending moments at column splices and tensile forces in interior columns;

(3) The peak bending moment at column splices is between 60% to 80% of flexural strength of the smaller column when

the maximum plastic hinge rotations are less than 0.04 radian for Category I and II system response, and reaches 80% and 90% of flexural strength of the smaller column when the maximum plastic hinge rotations are between 0.05 and 0.07 radian for Category III system response;

(4) With all of the general uncertainties inherent in these types of analyses considered, it would be reasonable to anticipate Category II system response when the frame is subjected to the design earthquake while Category III system response should be rare but not impossible. The strength demand on the column splice would be on the same order as that of the smaller column when the critical beam-to-column connection reaches its expected maximum deformation capacity, and that it does not appear unreasonable for the current seismic design provisions on the column splice in special moment frames to require the column splice to develop the strength of the smaller column.

Table 2 Summary of seismic response

Ground Motion Group	System Response Category	Peak System Response	Peak Demand on Column Splices
1	I	PHR[1]: $\approx$ 0.01 (20-story) $\approx$ 0.02 (9-story) $\approx$ 0.01 (4-story)	BM[2]: $\leq$ 0.50 (20-story) $\leq$ 0.50 (9-story) 0.20 to 0.35 (4-story)
2	II	PHR: $\approx$ 0.02 to 0.03 (20-story) $\approx$ 0.02 to 0.04 (9-story) $\approx$ 0.01 to 0.03 (4-story)	BM: 0.40 to 0.60 (20-story) 0.50 to 0.80 (9-story) 0.40 to 0.80 (4-story)
3	III	PHR: $\approx$ 0.05 to 0.07 (20-story) $\approx$ 0.05 to 0.07 (9-story) $\approx$ 0.04 (4-story)	BM: 0.60 to 0.80 (20-story) 0.70 to 0.90 (9-story) 0.50 to 0.80 (4-story)

1. PHR = Plastic Hinge Rotation at the beam end (Unit: radian); 2. BM = M_s/M_{pt}.

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