Span Length Effect on Seismic Demand on Column Splices in Steel Moment Resisting Frames

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SUMMARY
Steel moment frames have gone through intensive investigation to understand the cause of the damage since the 1994 Northridge earthquake. One major outcome of these research efforts was on the enhanced design requirements for column splices. For example, AISC 341 requires that column splices in intermediate and special moment frames, when not made using complete joint penetration 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 the spliced column. This paper discusses the effect of various span lengths on seismic demands on column splices in steel moment-resisting frames. A comprehensive nonlinear analytic investigation was undertaken to evaluate the seismic responses of 4- and 9- story moment-resisting frames with 6.10m (20ft), 7.62m (25ft) and 9.15m (30ft) span length, respectively, subjected to an ensemble of 20 strong ground motions. This study concludes that average peak story drift ratio and plastic hinge rotation increases, in general, as the span length decreases.

Keywords: Column splice, steel moment resisting frames, seismic design, various span lengths

1. INTRODUCTION

Since the 1994 Northridge earthquake, steel moment frames have gone through intensive investigation to understand the cause of the damage and to improve seismic design, construction, inspection, evaluation and retrofit of steel moment frames. These research efforts resulted in much improved understanding of seismic demand and capacity and design requirements for beam-to-column connections in steel moment frames, as well as enhanced requirements for column splices. For example, current and previous AISC 341 (2005, 2010) provisions 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.

Major issues in the seismic design of column splices can be listed as a) higher level of filler metal Charpy V-Notch toughness are required for welded column splices in moment frames of any type (special, intermediate, ordinary); b) CJP welds are required at column splices due to the high flexural and tensile demands (Shen *et.al.*, 2010).

Shen *et.al.* (2010) studied the seismic demand on column splices in special moment frames in an effort to address the question of whether the seismic design provisions as set forth by AISC 341 (2005, 2010) requiring CJP groove welds at column splices are justified, or unnecessarily conservative. They conducted nonlinear dynamic time history analyses on 4-, 9- and 20-story frames subjected to an ensemble of ground motions having a 2% probability of exceedance in 50 years. The main outcomes of their study are a) the seismic demand on column splices is related to the magnitude of plastic hinge rotations at beam ends; b) for low to moderate seismic responses, the bending moment demand at a splice closer to the column mid-height is lower than when the splice is taken at the beam-column joint centerline; c) for life-safety to near collapse seismic responses, the column splice location does not have any significant effect; d) for low to moderate and life-safety to near collapse seismic responses, the peak bending moment at a column splice reaches up to 60% and 70%-80% of the flexural strength of the smaller column; e) demand-to-capacity ratios may reach up to 0.8 when maximum plastic hinge rotations is around 0.02 rad and 0.9 to 1.0 when maximum plastic hinge rotation is about 0.07 rad in taller frames (above nine stories); f) low to moderate seismic responses can be expected from a special moment frame subjected to 2% probability of exceedance design earthquake. They also suggested that a) a significant margin of safety should be provided for the design of column-splices in special moment frames; b) for tall structures (above nine stories), current design requirements given by AISC 341 (2005 and 2010) requiring use of CJP welds seem reasonable, whereas this requirement seems conservative for shorter structures (less than nine stories).

This paper discusses the effect of various span lengths on seismic demands on column splices in steel moment-resisting frames. For this purpose, nonlinear dynamic time history analyses were carried out to evaluate the seismic response of 4- and 9- story moment-resisting frames previously studied by Shen *et.al.* (2010) with 9.15 span length. Two sets of additional 4- and 9-story frames were designed with 6.10m and 7.62m span lengths and the results were compared with those of the 9.15 span length.

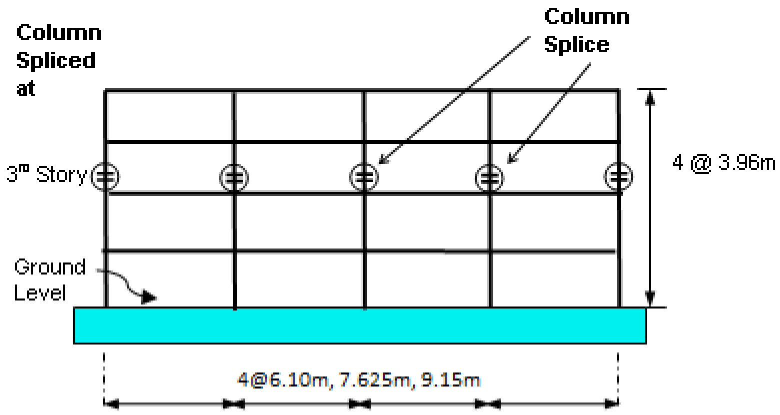
2. DESIGN OF 4- AND 9-STORY SPECIAL MOMENT FRAMES AND GROUND MOTIONS

Two typical steel moment frames with 4- and 9-stories, representing typical low- and medium-rise steel buildings were designed based on the seismic design requirements in ASCE 7 (2005) and AISC 341 (2005, 2010). To investigate the effect of span length, three different bays with 6.10m, 7.62m and 9.15 m were used (Fig. 2.1). The seismic demand on column splices in the 4- and 9-story buildings with 9.15m bays were previously investigated by Shen *et.al.* (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 both buildings is symmetrical. The 4-story building has plan dimensions of 54.90m x 24.40m, 30.48m and 36.60m corresponding to four 6.10m, 7.62m and 9.15 m bays, respectively. The typical story height is 3.96m and the columns are assumed to be fixed to the ground (Fig. 2.1a). The 9-story building has plan dimensions of 45.75m x 30.50m, 38.10m and 45.75m corresponding to five 6.10m, 7.62m and 9.15 m bays, respectively. The typical story height is 3.96m except at the ground and basement levels, where it is 5.50m and 3.65m, respectively (Fig. 2.1b). The columns of the 9-story buildings are assumed to be pinned at the lowest basement level and they run continuously through the ground level framing. For the 9-story building, concrete foundation walls and surrounding soil are assumed to prevent any significant horizontal displacement of the structure at the ground level, i.e. the seismic base is assumed to be at the ground level.

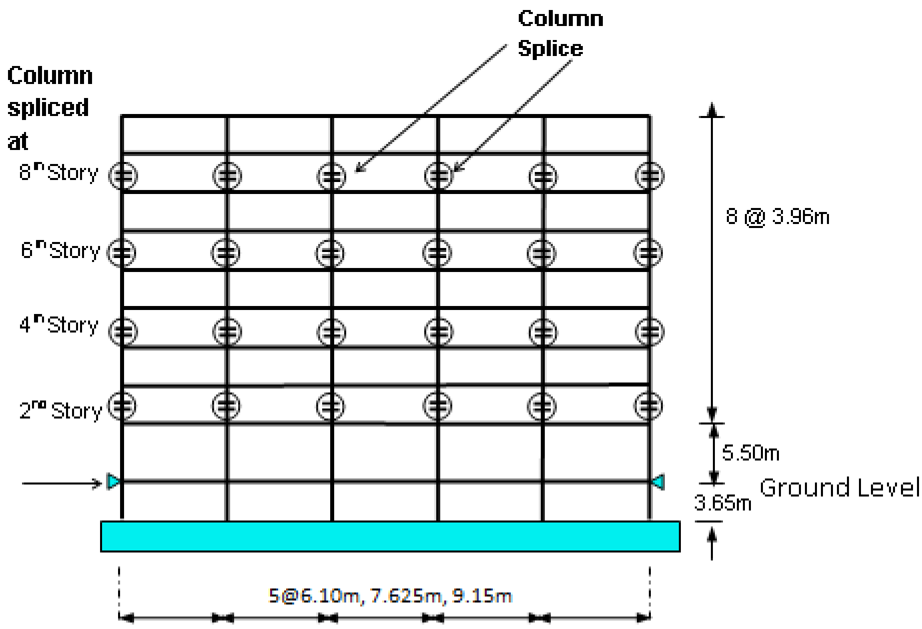
The 4- and 9-story buildings were designed for a site in downtown Los Angeles, where S_s is 2.0g and S_1 is 1.0g. The special moment frames corresponding to the 4- and 9-story buildings were designed using response modification factor of R = 8. The ASCE 7 (2005) base shears for the 4-story frame with 6.10m, 7.62m and 9.15m bays were found to be 4,368kN, 5,471kN and 6,405kN, respectively. The ASCE 7 (2005) base shears for the 9-story frame with 6.10m, 7.62m and 9.15m bays were 5,409kN, 6,761kN and 8,674kN, respectively. Detailed information about the design of the frames can be found in Shen *et.al.* (2012).

The column splice location in Shen *et.al.*'s (2010) study with 9.15m bays was considered to be varying for two bounding cases, PC and SC. PC refers to the column splice location 1.22m (4ft) above the finished floor elevation as suggested in Section D2.5a of AISC 341 (2010), whereas SC refers to the column splice location 1.22m (4ft) from the beam centerline AISC 341 (2005 and 2010). In this study, the column splices are assumed to be at 1.22m (4ft) above the finished floor elevation. The columns in the 4-story frame were spliced only at its third floor, whereas the column splices in the 9-story frame were located at every second floor.

A total of 20 ground motion records, named as LA21 to LA40 were used in this study. The ground motions are the same as those used in Shen *et al.* (2010). These ground motions were used in a FEMA-sponsored research project on steel moment frames damaged in the 1994 Northridge earthquake and identified as having a 2% probability of exceedance in 50 years by SAC. The acceleration time histories of the ground motions were derived from historical recordings or from physical simulations and altered so that their mean response spectrum matches the 1997 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 (Sommerville *et.al.*, 1997).



a. 4-story frame



b. 9.story frame

Figure 2.1. Elevation of the 4- and 9-story frames

3. SEISMIC RESPONSE OF THE FRAMES

Inelastic dynamic time history analyses were carried out to study the span length effect on column splices of the 4- and 9-story frames. The frames were subjected to 20 ground motion accelerations (LA21 to LA40). The frames were modeled as beam and column elements with 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.

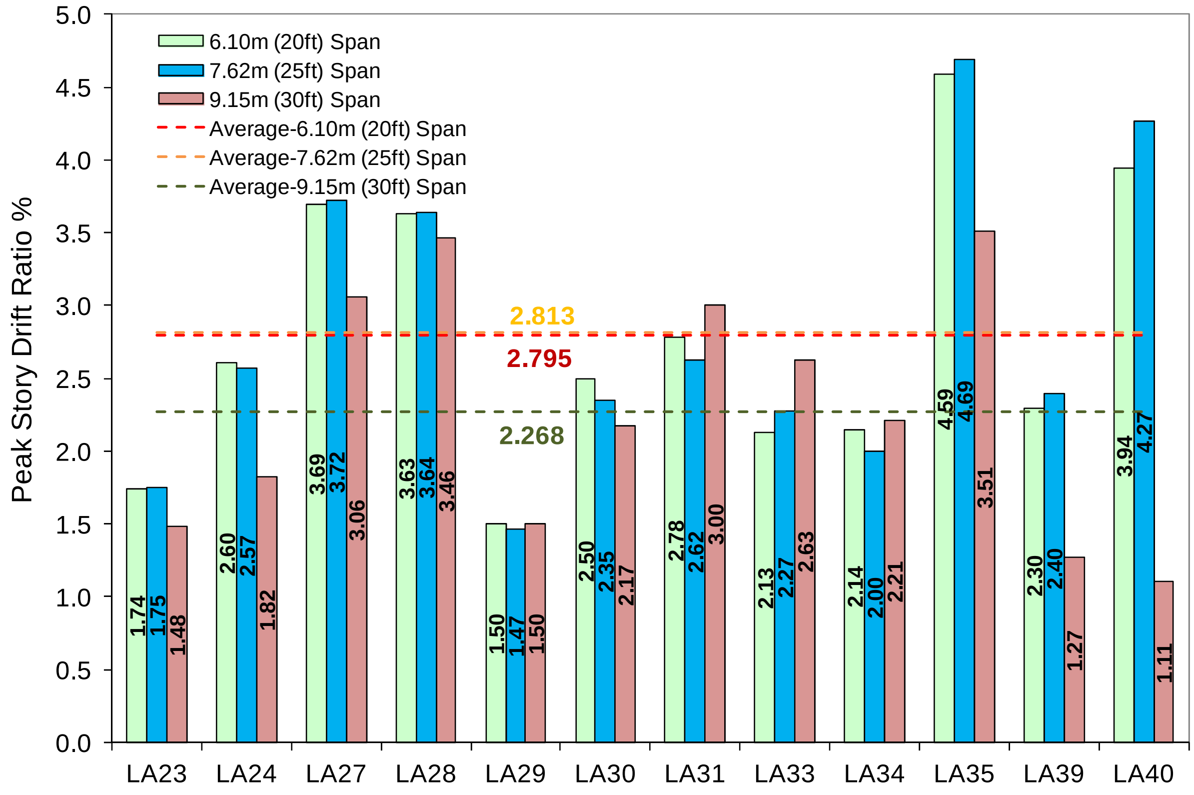
Two groups of response parameters are selected for the seismic response evaluation of the column splices: (1) peak story drift ratios and peak plastic hinge rotations at beam ends; (2) (a) the peak bending moment at the column splice, M_s, normalized by the plastic moment of the smaller column (on the top of the splice), M_{pt}; and (b) the peak 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 response parameters in the first group provides information about the seismic performance of the frame as a whole for a given ground motion and can be considered as the system response parameters. The second group of response parameters 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 (Shen *et.al.*, 2010).

Shen *et.al.* (2010) classified the ground motions for each individual frames based on the seismic response of the frames in terms of peak story drift and plastic hinge rotation at beam ends. They divided the ground motions from LA21 to LA40 into three ground motion groups (GMG 1, GMG 2 and GMG 3) for both frames based on increasing peak story drifts and plastic hinge rotation at beams. The above-mentioned seismic response parameters corresponding to the GMG 1 ground motions are presented in this study. The seismic responses of the 4- and 9-story frames subjected to GMG 1 ground motions are considered to be mild responses with story drift ratios from 1% to 2% and plastic hinge rotations from 0.01 to 0.02. The structures in this category are considered to experience functional to moderate structural damage with limited inelastic deformation in a small number of beams having less than 0.02 plastic hinge rotations in any beam. The ground motions in the GMG 1 are LA23, LA24, LA27, LA28, LA29, LA30, LA31, LA33, LA34, LA35, LA39, LA40 and LA23, LA29, LA30, LA39 for the 4- and 9-story frames, respectively. The results for the other ground motion groups (GMG 2 and GMG 3) can be found in Shen *et.al.* (2012).

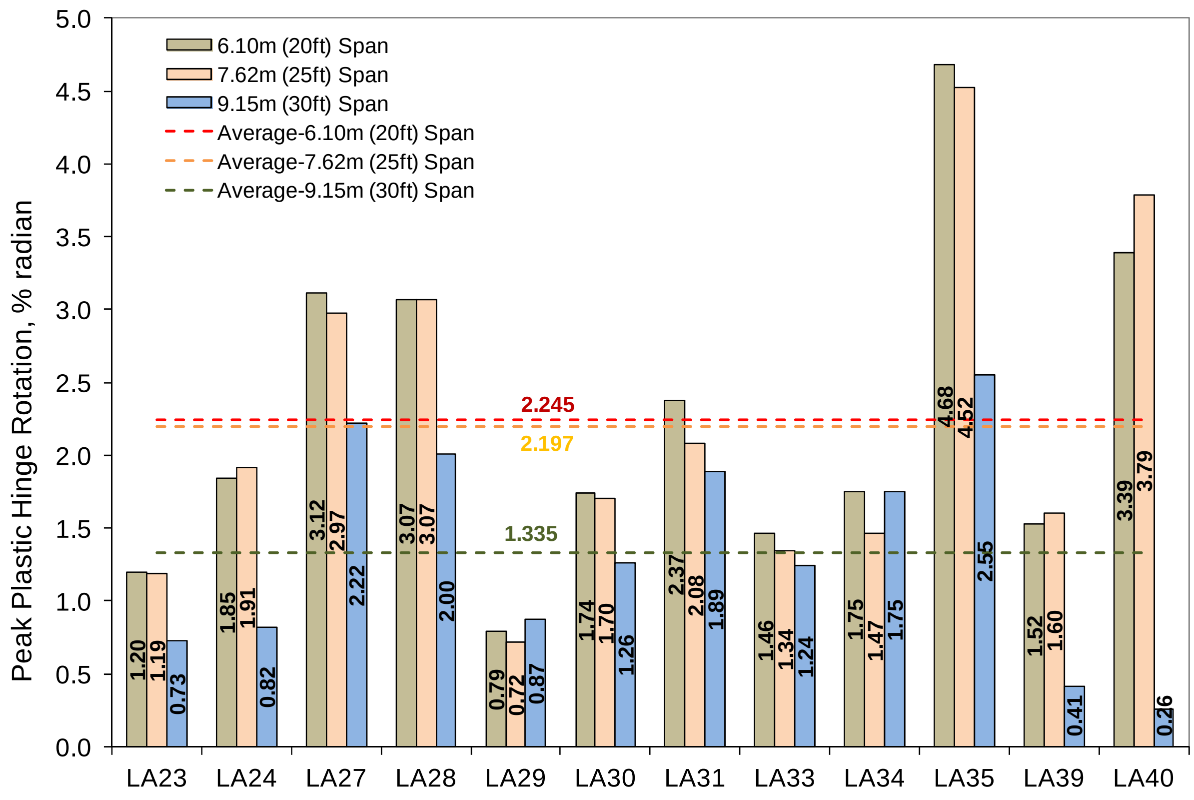
3.1. Peak Story Drifts and Plastic Hinge Rotations

Figs. 3.1 and 3.2 show the system response parameters in terms of peak story drift ratio and peak plastic hinge rotation at beam ends in the 4- and 9-story frames subjected to GMG 1 ground motions, respectively. For the 4-story frame, average peak story drift ratios increased about 23% for the frames with 7.62m and 6.10m bays compared to that of with 9.15m bay (Fig. 3.1a). For the 9-story frame, average peak story drift ratios increased about 20% and 12% for the frames with 7.62m and 6.10m bays, respectively, compared to that of with 9.15m bay (Fig. 3.2a).

For the 4-story frame, average peak plastic hinge rotations increased about 64% and 68% for the frames with 7.62m and 6.10m bays compared to that of with 9.15m bay (Fig. 3.1b). For the 9-story frame, average peak plastic hinge rotations increased about 14% and 4% for the frames with 7.62m and 6.10m bays compared to that of with 9.15m bay (Fig. 3.2b). For both frames, the average peak plastic hinge rotations were observed to be on the order of 2%, which is considered to be in the mild response category.

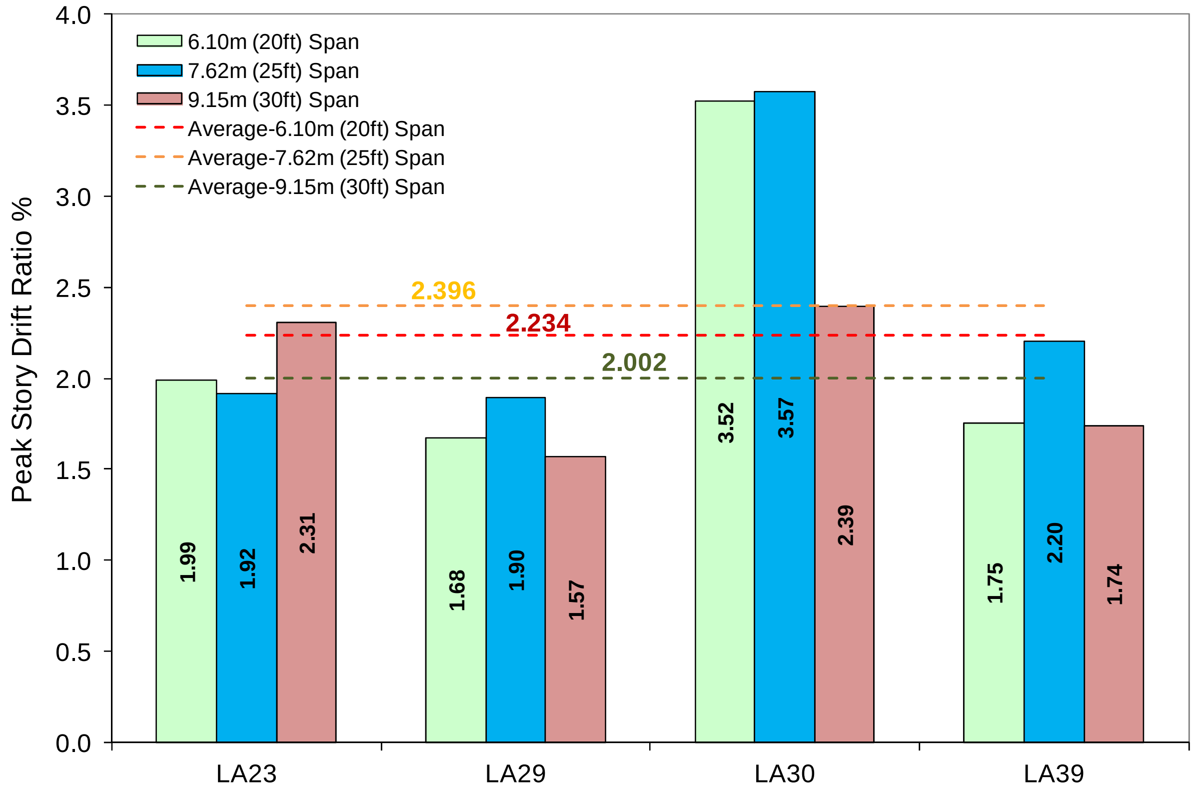


a. Peak story drift ratio

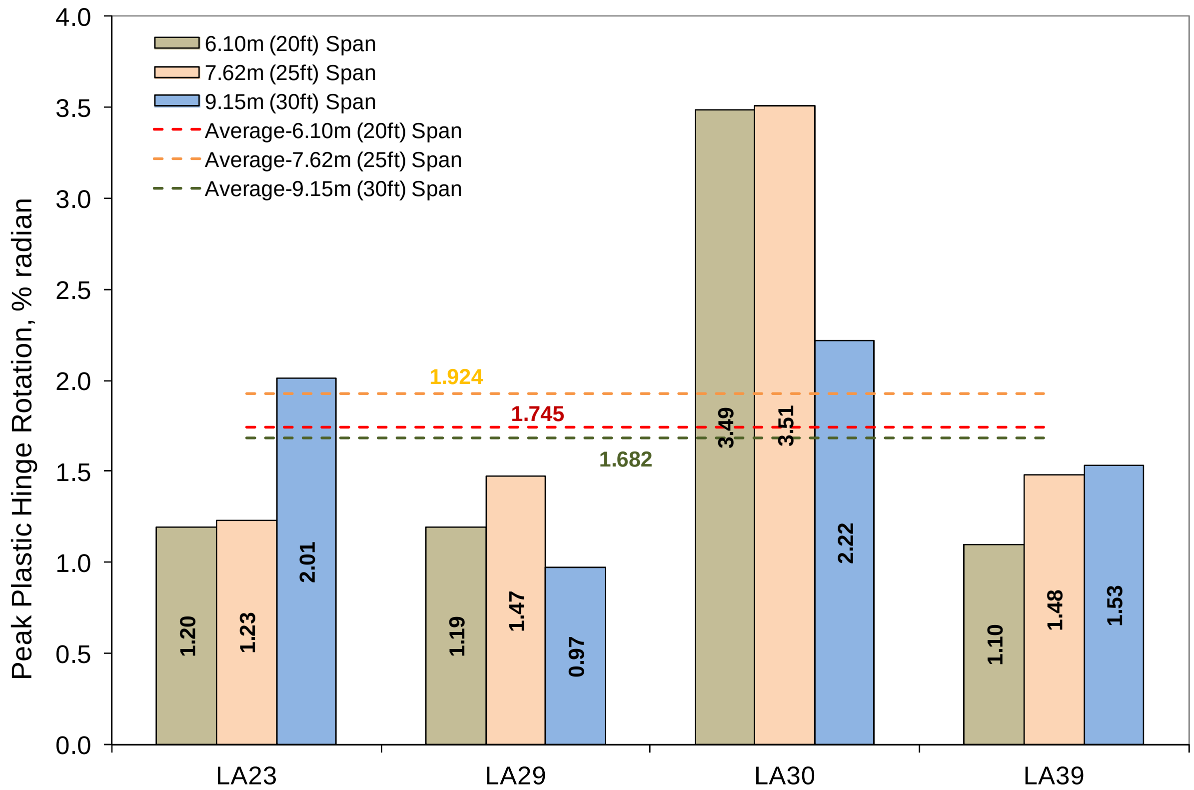


b. Peak plastic hinge rotation at beam ends

Figure 3.1. System response of the 4-story frame subjected to GMG 1 ground motions



a. Peak story drift ratio



b. Peak plastic hinge rotation at beam ends

Figure 3.2. System response of the 9-story frame subjected to GMG 1 ground motions

3.2. Bending Moment Demand on Column Splices

Figs. 3.3a and 3.4a present the peak bending moment, M_s, in all column splices in the 4- and 9-story frames, respectively, normalized by M_{pt}, the plastic moment of the smaller column on the top of splice subjected to GMG 1 ground motions. For the 4-story frame, the average peak value of M_s/M_{pt} increased about 54% for the frames with 7.62m and 6.10m bays compared to that of with 9.15m bay (Fig. 3.3a).

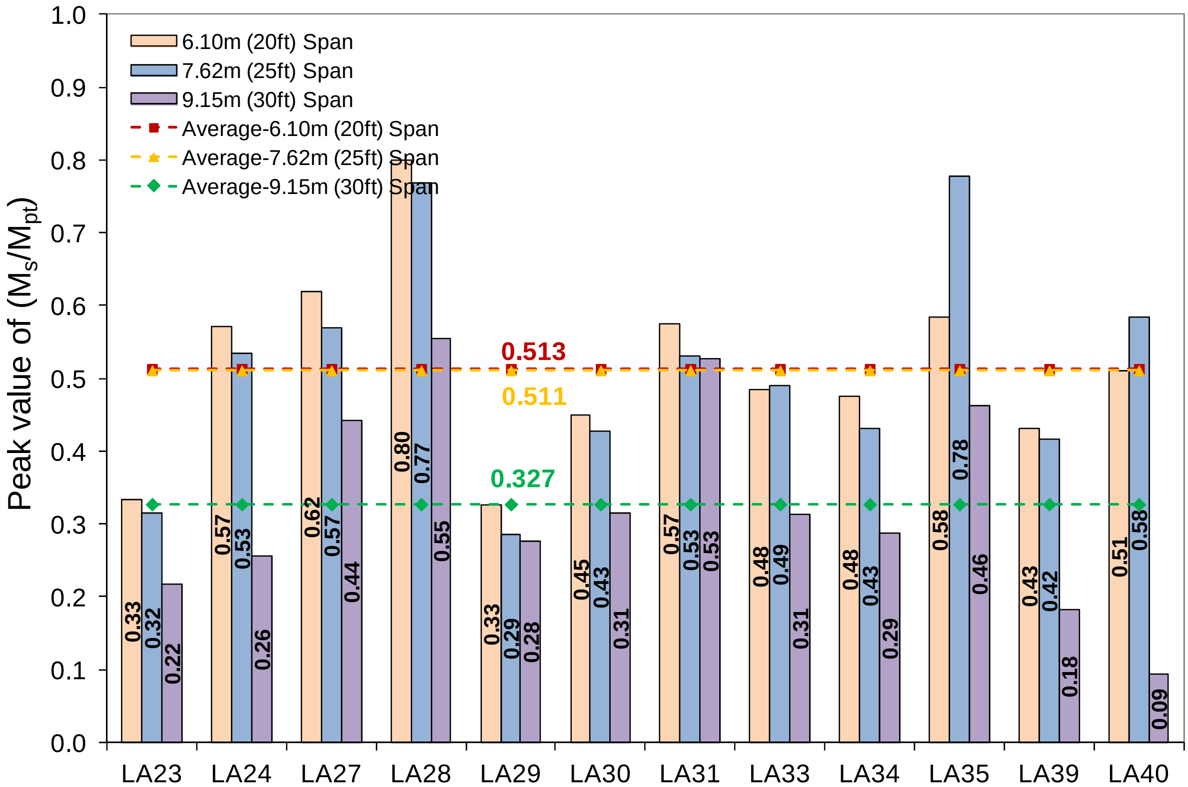
Similar results were observed for the 9-story frame, whereas the average peak value of M_s/M_{pt} increased about 47% for the frames with 7.62m and 6.10m bays compared to that of with 9.15m bay (Fig. 3.4a). Suprisingly, the average peak value of M_s/M_{pt} remained almost the same for the frames with 7.62m and 6.10m bays. However, it should be noted that the M_s/M_{pt} ratio at the column splice was observed to be less than 0.40 for the frames with 9.15m bay, but increased up to 0.50 for the frames with 7.62m and 6.10m bay. It should be noted that GMG 1 ground motions refer to mild response category with little or limited inelastic response.

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

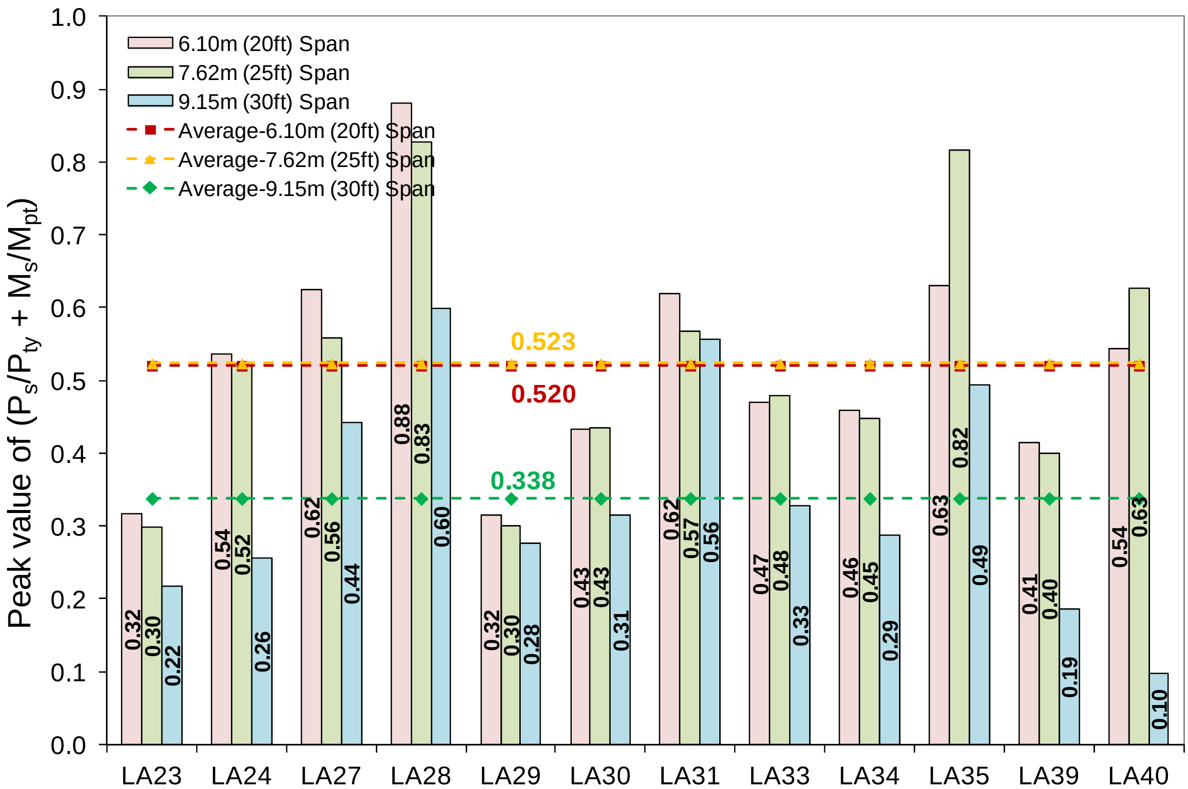
Peak combination of bending moment and axial tensile force in column splices is also investigated, because axial tensile forces can cause significant demands on column splices, especially, in a taller frame. Figs. 3.3b and 3.4b show the normalized bending moment and axial tensile force (P_s/P_{ty} + M_s/M_{pt}) in column splices in the 4- and 9-story frames, respectively, where P_s is the tensile force at the splice, and P_{ty} is the nominal tensile strength of the smaller column. It should be noted that axial force demand at the column splice is independent of splice location. Thus, in cases where axial force is significant, combination of bending moment and axial tensile force in the column splice will not be affected by the splice location significantly.

For the 4-story frame, the average peak value of $P_s/P_{ty} + M_s/M_{pt}$ increased about 54% for the frames with 7.62m and 6.10m bays compared to that of with 9.15m bay (Fig. 3.3b). For the 9-story frame, the average peak value of $P_s/P_{ty} + M_s/M_{pt}$ increased about 63% and 54% for the frames with 7.62m and 6.10m bays, respectively, compared to that of with 9.15m bay (Fig. 3.4b). For both frames with 9.15m bays, peak value of $P_s/P_{ty} + M_s/M_{pt}$ remained almost the same, 0.338 and 0.339 for the 4- and 9-story frames, respectively. The same conclusion applies for the frames with 7.62 and 6.10m bays (Figs. 3.3b and 3.4b).

Tensile axial forces in columns in the 4- and 9-story frames were not observed to be significant for GMG 1 ground motions (Figs. 3.3b and 3.4b). However, they were significant when both frames were subjected to GMG 2 and GMG 3 ground motions (Shen *et.al.*, 2012).

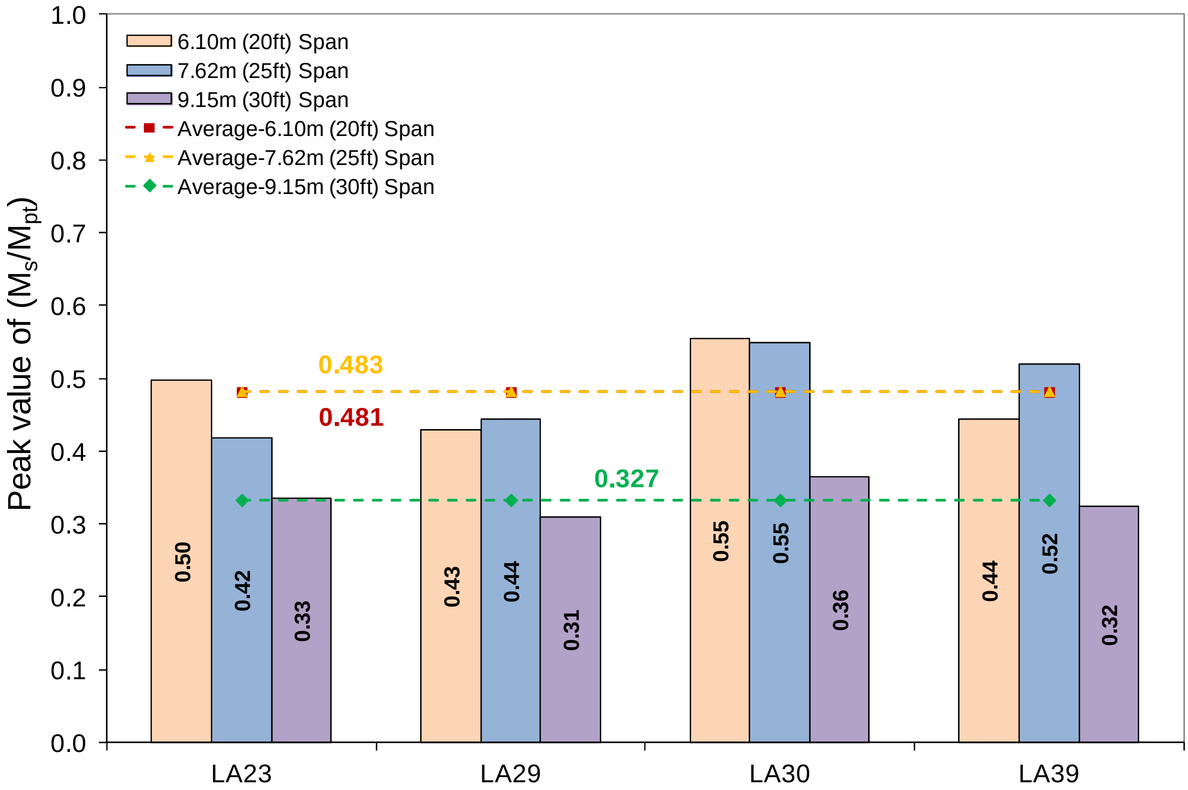


a. Peak value of M_s/M_{pt}

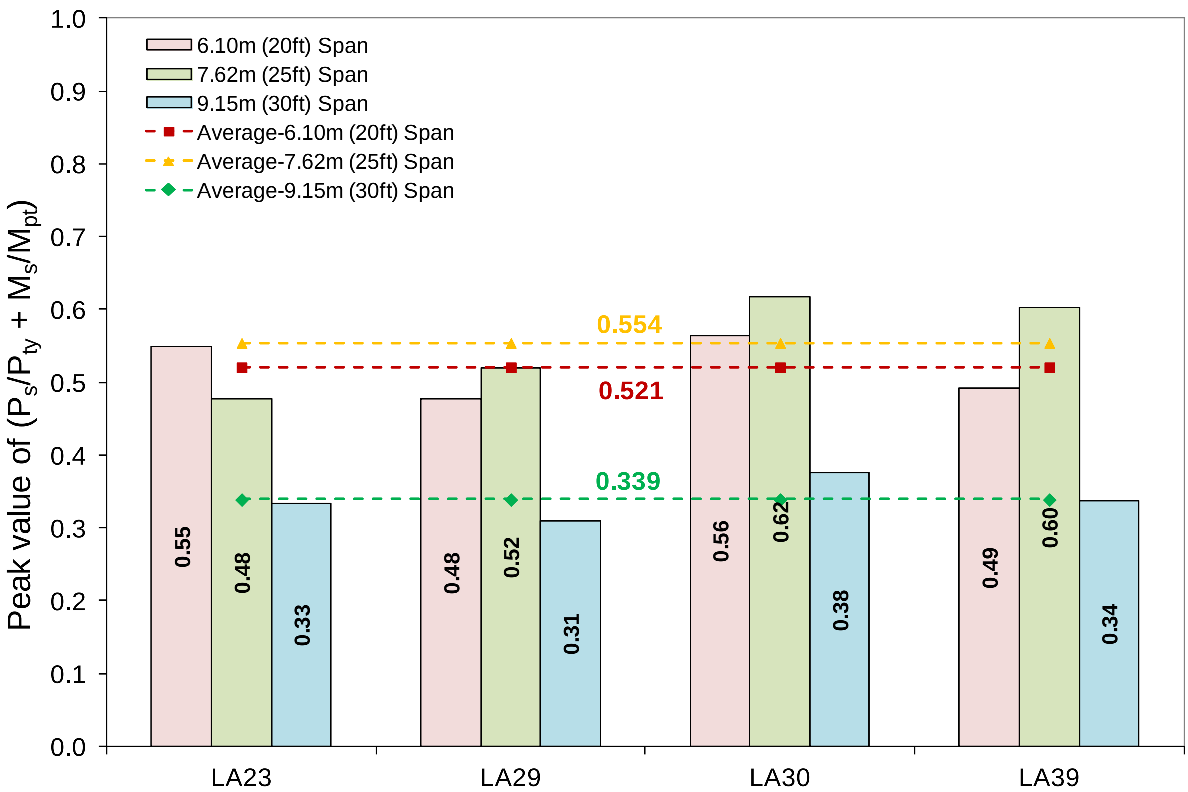


b. Peak value of $(P_s/P_{ty} + M_s/M_{pt})$

Figure 3.3. Columns splice response of the 4-story frame subjected to GMG 1 ground motions



a. Peak value of M_s/M_{pt}



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

Figure 3.4. Columns splice response of the 4-story frame subjected to GMG 1 ground motions

4. CONCLUSIONS AND RECOMMENDATIONS

For the 4- and 9-story steel moment frames investigated in this study, column splice demand based on a structural model assuming the column splice located four feet above the finished floor elevation, the following conclusions can be drawn and recommendations can be made:

(1) Average peak story drift ratios increase, in general, as the span length decreases.
(2) Average peak plastic hinge rotations increase as the span length decreases. This increase is, especially, significant for shorter frames, but average peak plastic hinge rotations remain about 2% for both 4- and 9-story frames.
(3) Average peak value of M_s/M_{pt} remains almost the same for both 4- and 9-story frames with 7.62m and 6.10m bays, i.e. span length does not affect the M_s/M_{pt} ratio significantly in low- to mid-rise frames with span length less than 7.62m.
(4) The peak value of M_s/M_{pt} ratio at the column splice is, in general, expected to be less than 0.40 for the low- to mid-rise frames with 9.15m bay, but can increase significantly up to 0.50 for the frames with 7.62m and 6.10m bay.
(5) It is recommended that a significant margin of safety be provided for column splices in special moment frames due to the uncertainty inherent in reliably estimating the capacity of a welded column splice constructed using partial joint penetration groove welds.
(6) For tall special moment frames (above nine stories), current design requirements mandating use of complete joint penetration groove welds in welded column splices appears reasonable, because the tensile axial loads on column splices can be significant.
(7) For short special moment frames (less than or equal to nine stories), current design requirements mandating use of complete joint penetration groove welds in welded column splices appear conservative.

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