Effects of column splice properties on seismic demands in steel moment frames

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ABSTRACT: Dynamic inelastic response history analysis of a 2-D model of a nine storey steel frame were carried out with different column splice strengths and stiffnesses using a suite of 20 MCE level earthquake records. Splices were located every 2nd storey at one third of the storey height up from the column below. It was shown that (i) the presence of even very flexible splices increased the frame period by less than 3%, (ii) flexible splices increased storey drift ratios by up to 27%, (iii) splice stiffnesses of zero to infinity had no effect on frame displacements, (iv) the splice moment demand increased with increasing splice stiffness on the frame and was as high as 99% of the column flexural capacity.

1 INTRODUCTION

Structural damage recently observed in Northridge earthquake reveals that connections in steel structures may be vulnerable to failure (FEMA, 2000a). The column splice is one of these connections. They are essential in multi-story construction due to limitations on carrying sections to sites, producing long enough members for multi storey buildings and the economical advantage of reducing section size with height.
Current NZS 3404 design specifications for column splices, in frames required to resist significant seismic forces (i.e. Category 1 and 2 frames), require the connection to provide 50% of the reduced flexural strength of the smaller column as well as 25% of its design shear capacity. For columns in frames subject to lower seismic actions (i.e. Category 3 and 4 frames) less flexural and shear capacity is prescribed, i.e. 30% and 15% of the moment and shear capacity of the smaller section respectively. Splices also should be designed for 50 percent of the member compressive or tensile capacity as appropriate. Contact splices in columns subjected to axial compression which are part of associated structural system, but not seismic resisting system, should be designed for 15% of shear and compression capacity of column. Non-contact splices should be designed for the same shear force but for higher compression capacity (i.e. 30% of axial compression design capacity) (NZS3404, clauses 9.1.4.1-c, 12.9.2.2, 12.9.2.3). Furthermore, it recommends splice placement within the middle third of the column along which moment is considered to be very small (NZS3404, clause 12.9.6). However, research studies have shown that in the cases where higher modes of the structure are stimulated, especially in both seismic and gravity columns in high rise buildings, the common pattern of moment distribution along columns will change during the earthquake excitation and significant splice moments may develop that should be accounted for. Consequences of splice failure may result in the upper column moving relatively to the lower column. This could have disastrous consequences especially if there are many stories above the splice level considered.

Also, there is not any specific provision for the required *stiffness* of splices and this may affect overall frame performance. If splices are strong enough to carry the demand but not sufficiently stiff, they may exhibit large deformations at a certain level of strength. Since splices are generally placed at the same height up the structure, there is an increased probability of large drifts due to a frame partial height sway as shown in Figure 1.

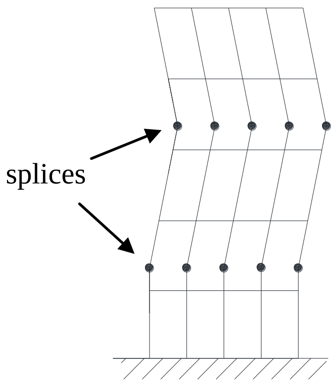


Figure 1. Possible partial sway mechanism of a frame due to low column splice stiffness/strength

Splice rotation is more likely to occur in bolted rather than welded splices but this effect has been ignored in the research conducted so far; Popov et al. (1989), Bruneau et al. (1990), Shen et al. (2008), Shen et al. (2010), Akbas et al. (2011). In this study, the effects of splice strength and stiffness on moment demand of column splices, drift and displacement responses of a nine story building are investigated. In particular, answers are sought to the following questions:

i. What effect do splices have on the frame period?

ii. How do flexible splices affect peak frame drifts?

iii. What is the likelihood of splice yielding?

2 FRAME AND EARTHQUAKES PROPERTIES

The structure under investigation is a 9-story steel moment frame from the SAC steel project which represents a mid-rise frame. Twenty ground motions from SAC steel project, named LA21 to LA40 are used in the analyses.

2.1 Properties of the frame and splices

Figure 2 illustrates details of the 2D steel moment frame and the location of splices. Splices are modelled as rotational springs which located in the lower bound of the code prescribed range, i.e. 34 percent of story-height above the beam level. The exterior column, to which beams are connected by pinned joints, is oriented about its minor axis. Member sizes are also listed in Table 1.

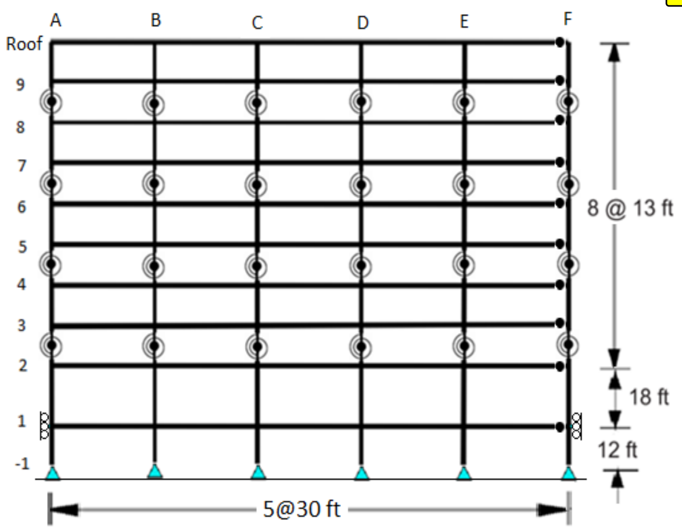


Figure 2. Details of 9 story 2-D frame

Table 1. Details of 9 story frame elements (Gupta et al., 1999)

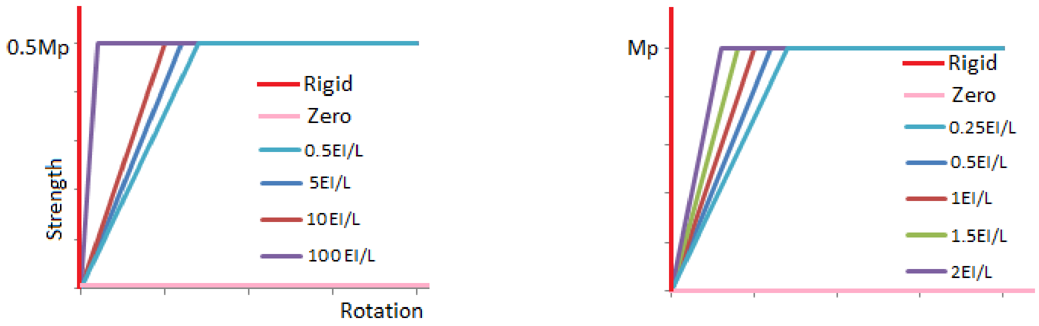
story/Floor	COLUMNS		DOUBLER PLATES(in)	GIRDER
	Exterior	Interior		
-1/1	W14X370	W14X500	0.0	W36X160
1/2	W14X370	W14X500	0.0	W36X160
2/3	W14X370, W14X370	W14X500, W14X455	0.0	W36X160
3/4	W14X370	W14X455	0.0	W36X135
4/5	W14X370, W14X283	W14X455, W14X370	0.0	W36X135
5/6	W14X283	W14X370	0.0	W36X135
6/7	W14X283, W14X257	W14X370, W14X283	0.0	W36X135
7/8	W14X257	W14X283	0.0	W30X99
8/9	W14X257, W14X233	W14X283, W14X257	0.0	W27X84
9/Roof	W14X233	W14X257	0.0	W24X68

* Column A has exterior column section oriented about strong axis.

** Column F has exterior column section oriented about weak axis.

*** Columns B, C, D and E have interior column sections.

A parametric study was conducted to quantify splice stiffness and strength effects on frame response. The splice yielding strength is assumed to be equal to either 50% and 100% of plastic moment capacity of the smaller column at the location of splice. The behaviour of splices are considered to be elastic perfectly plastic and their stiffness is defined to change according to diagrams in Figure 3. The stiffness of splices selected is based on the limited literature available and the range was broadened for the study to also consider extreme stiff elastic and flexible scenarios as benchmark cases. Actual stiffnesses will be available for experimental results in the near future. Simulations were carried out in OpenSees and nonlinear behaviour of frame elements was modelled with fiber hinges.



a) Splice strength of 50% of moment capacity

b) Splice strength of 100% of moment capacity

Figure 3. Rotational characteristics of splices

2.2 **Properties of earthquake motions**

Twenty ground motion records were applied to the frame. These are quite big earthquakes representing MCE level with 2% possibility of occurrence in 50 years. Table 2 presents the properties of the ground motions.

Table 2. Properties of earthquake ground motions

SAC Name	Record	Earthquake Magnitude	Distance (km)	Scale Factor	Number of Points	DT (sec)	Duration (sec)	PGA (cm/sec^2)
LA21	1995 Kobe	6.9	3.4	1.15	3000	0.02	59.98	1258
LA22	1995 Kobe	6.9	3.4	1.15	3000	0.02	59.98	902.75
LA23	1989 Loma Prieta	7	3.5	0.82	2500	0.01	24.99	409.95
LA24	1989 Loma Prieta	7	3.5	0.82	2500	0.01	24.99	463.76
LA25	1994 Northridge	6.7	7.5	1.29	2990	0.005	14.945	851.62
LA26	1994 Northridge	6.7	7.5	1.29	2990	0.005	14.945	925.29
LA27	1994 Northridge	6.7	6.4	1.61	3000	0.02	59.98	908.7
LA28	1994 Northridge	6.7	6.4	1.61	3000	0.02	59.98	1304.1
LA29	1974 Tabas	7.4	1.2	1.08	2500	0.02	49.98	793.45
LA30	1974 Tabas	7.4	1.2	1.08	2500	0.02	49.98	972.58
LA31	Elysian Park (simulated)	7.1	17.5	1.43	3000	0.01	29.99	1271.2
LA32	Elysian Park (simulated)	7.1	17.5	1.43	3000	0.01	29.99	1163.5
LA33	Elysian Park (simulated)	7.1	10.7	0.97	3000	0.01	29.99	767.26
LA34	Elysian Park (simulated)	7.1	10.7	0.97	3000	0.01	29.99	667.59
LA35	Elysian Park (simulated)	7.1	11.2	1.1	3000	0.01	29.99	973.16
LA36	Elysian Park (simulated)	7.1	11.2	1.1	3000	0.01	29.99	1079.3
LA37	Palos Verdes (simulated)	7.1	1.5	0.9	3000	0.02	59.98	697.84
LA38	Palos Verdes (simulated)	7.1	1.5	0.9	3000	0.02	59.98	761.31
LA39	Palos Verdes (simulated)	7.1	1.5	0.88	3000	0.02	59.98	490.58
LA40	Palos Verdes (simulated)	7.1	1.5	0.88	3000	0.02	59.98	613.28

3 BEHAVIOUR

3.1 Pushover analyses

Pushover analyses were performed for the frames with different splice flexibility and strength. An inverted triangle (first mode) lateral load pattern was applied to the frame. Base shear force versus roof drift ratio is displayed in Figure 4. There is not a significant difference in the behaviour of frames with non-zero splice stiffnesses. Although frame with pinned splices exhibits similar behaviour up to 2% of drift ratio, its pushover curve deviates from other curves followed by a negative post yielding stiffness. Analyses were also repeated for the frames with 50% splice capacity. It was observed that strength doesn't have any effect on the behaviour of frames since the splice strength was not reached.

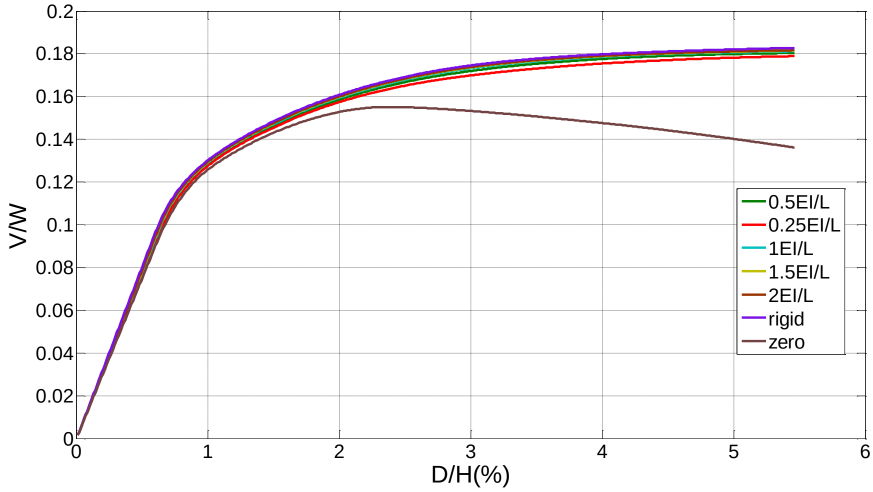


Figure 4. Pushover curves of frames with different splice stiffness

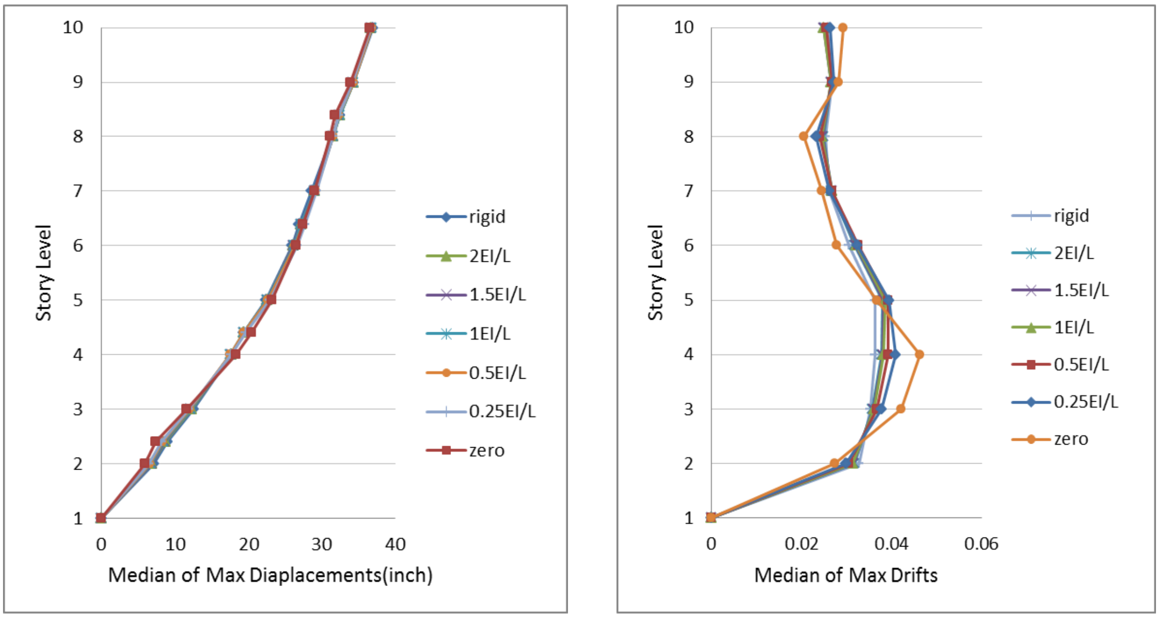
3.2 Time history analyses

Table 3 shows that the first and second mode periods increase by less than 3% and 5% respectively as the splice stiffness increases from fully rigid to fully pinned. Although it is not very significant for this model period changes are greater if the splices are located closer to floors.

Table 3. First and second mode period of the frame with different column splice stiffness

Splice Stiffness	Zero	0.25EI/L	0.5EI/L	1EI/L	1.5EI/L	2EI/L	5EI/L	10EI/L	100EI/L	Rigid
First Mode Period	2.37	2.35	2.34	2.33	2.32	2.32	2.31	2.30	2.30	2.30
Second Mode Period	0.90	0.89	0.89	0.88	0.88	0.87	0.87	0.87	0.86	0.86

Figure 4a shows that the story maximum displacements from the suite of ground motion records are almost the same for all splice stiffnesses and the response is not sensitive to splice stiffness. More flexible splices generally cause greater drift ratios at the lower stories, with a change in drift as high as 27% as shown in Figure 4b.



a) Displacements b) Drift ratios

Figure 5. Frame median peak displacement and drift ratios for different splice stiffness

(Moment capacity is 100% of the plastic capacity of the smaller section, MCE level records)

The moment ratio (M/M_p) at the splice location increased with splice stiffness as shown in Table 4. For exterior Column F, the moment demand is higher compared to other columns of the frame, with the highest median moment occurring in the rigid splice of 0.99M_p.

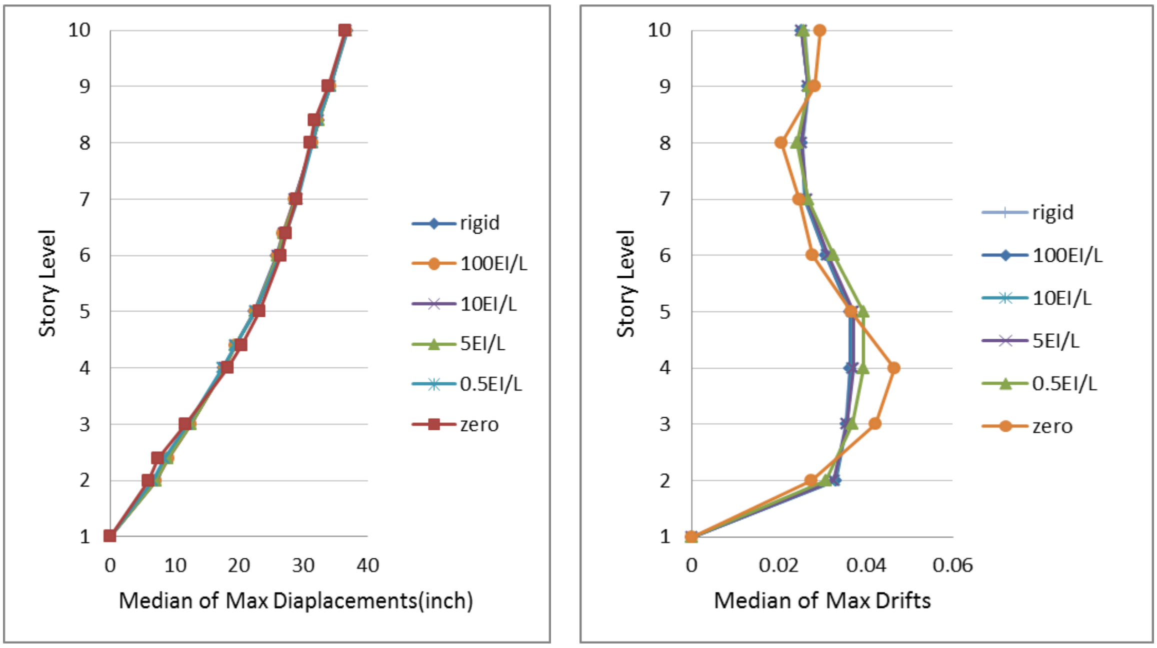
Table 4. Median of maximum moment ratio (M/M_p) at the location of splices

(Moment capacity is 100% of the plastic capacity of the smaller section, MCE level records)

Stiffness	**Column A**				**Interior columns**				**Column F**			
	3rd floor	**5th floor**	**7th floor**	**9th floor**	**3rd floor**	**5th floor**	**7th floor**	**9th floor**	**3rd floor**	**5th floor**	**7th floor**	**9th floor**
Zero	0	0	0	0	0	0	0	0	0	0	0	0
0.25EI/L	0.11	0.10	0.10	0.10	0.15	0.12	0.11	0.12	0.12	0.11	0.11	0.10
0.5EI/L	0.16	0.15	0.15	0.15	0.21	0.19	0.17	0.17	0.20	0.18	0.20	0.17
1EI/L	0.20	0.21	0.20	0.20	0.28	0.25	0.23	0.23	0.30	0.29	0.33	0.28
1.5EI/L	0.21	0.23	0.22	0.22	0.27	0.26	0.24	0.23	0.36	0.37	0.42	0.36
2EI/L	0.22	0.25	0.23	0.24	0.33	0.30	0.30	0.29	0.41	0.42	0.49	0.41
Rigid*	0.26	0.30	0.29	0.29	0.38	0.35	0.41	0.36	0.74	0.83	0.99	0.85

* Splices are rigid elastic. No yielding occurs in splices at M_p.

Analyses were also carried out for the frame with splice capacity of 50% of the column plastic moment capacity. Figure 6 shows the displacement and drift ratios. For all splice stiffnesses, the displacement response hardly changed because the response was not very sensitive to splice strength. Splice stiffnesses more than 5EI/L generate drift ratios almost equal to that of a frame with the rigid elastic splices as shown in Figure 6b.



a) Displacements b) Drifts ratios

Figure 6. Frame median peak displacement and drift ratios for different splice stiffness

(Moment capacity is 50% of the plastic capacity of the smaller section, MCE level records)

Moment ratios are presented in Table 5. For the case when the splice stiffness is 0.5EI/L, the moment ratio can be seen to be identical to that of Table 4 indicating no splice yielding. Also, the moment ratio does not seem to be affected significantly by splice stiffnesses when it is greater than 5EI/L.

Table 5. Median of maximum plastic moment ratio (M/M_p) at the location of splices

(Moment capacity is 50% of the plastic capacity of the smaller section, MCE level records)

Stiffness	Column A				Interior columns				Column F			
	3rd floor	5th floor	7th floor	9th floor	3rd floor	5th floor	7th floor	9th floor	3rd floor	5th floor	7th floor	9th floor
Zero	0	0	0	0	0	0	0	0	0	0	0	0
0.5EI/L	0.16	0.15	0.15	0.15	0.21	0.19	0.17	0.17	0.20	0.18	0.20	0.17
5EI/L	0.24	0.28	0.27	0.26	0.36	0.34	0.36	0.33	0.46	0.45	0.48	0.49
10EI/L	0.25	0.29	0.29	0.27	0.38	0.35	0.38	0.34	0.48	0.47	0.48	0.49
100EI/L	0.25	0.30	0.30	0.29	0.38	0.36	0.40	0.36	0.49	0.48	0.49	0.50

4 CONCLUSIONS

Nonlinear time history and pushover analyses for a mid-rise frame have been conducted using MCE level ground motions. Column splices have been explicitly considered in the model as rotational springs. Effects of strength and stiffness of splices on the responses of structure has been discussed in this paper. Conclusions are presented as following:

1- Rotational stiffness of column splices located at one third of column did not affect the period of the frame significantly.

2- For the frames with all splice stiffnesses, the roof maximum displacements from the suite of ground motion records were almost the same. More flexible splices generally caused greater drift ratios, with a change in drift as high as .

3- Strength of splices did not have any effect on displacement and drift responses if developing ductile behaviour.

4- The median ratio of splice moment demand to the plastic moment capacity of the smallest member at the splice is affected by splice stiffness. This median ratio reached 0.99 for rigid splices. This implies little or no splice yielding if the splice strength is 100% of the strength of the smaller member, but significant yielding is expected for frames with a splice strength of 50% of the member capacity. Less severe demands would be observed with design level, rather than MCE level ground motion records.

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