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EFFECTS OF COLUMN SPLICE LOCATION ON SEISMIC DEMANDS IN STEEL MOMEMNT FRAMES CONSIDERING SPLICE FLEXIBILITY

Tork Ladani F.*, MacRae G.A.* and Chase J.G.*

* University of Canterbury
e-mails: Fahimeh.torkladani@pg.canterbury.ac.nz, Gregory.macrae@canterbury.ac.nz
Geoff.chase@canterbury.ac.nz

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Abstract. *This study aims to quantify the effect of splice connections in columns of steel frames on the responses for DBE and MCE level of ground motions. Inelastic dynamic time history analysis was conducted to assess splice and frame demands. Splices were characterized by strength and stiffness and were explicitly considered in the analyses. It was shown that (1) the presence of flexible splices increased the frame first and second mode periods by about 2% and 4%, respectively; (2) non-zero flexible splices increased storey drift ratios by up to 11%; (3) splice stiffnesses had almost no effect on frame displacements; (4) the splice moment demand increased with increasing splice stiffness on the frame and was as high as 47% of the column flexural capacity; (5) shear demand on splices can reach 48% of the nominal shear capacity; and (6) splice location can affect the demands of the frame and splices.*

1 INTRODUCTION

Structural damage observed in the Northridge earthquake reveals that connections in steel structures may be vulnerable to failure [1]. 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 advantages obtained from reducing section size with height.

In US standards, the latest steel seismic design provisions, AISC 341-10 [2] clauses E2.6g and E3-6g, stipulate design flexural capacity of column splices in special and intermediate frames to be at least equal to the flexural strength of the smaller column. The shear strength of the splice is prescribed to satisfy the demand associated with the flexural hinging at both ends of the column assuming double curvature deflection. Current NZS 3404 design specifications [3] 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 design 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, stating 30% and 15% of the moment and shear design capacity of the smaller section respectively. Splices also should be designed for 50 percent of the member compressive or tensile design 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 design 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 taken into account. Consequences of splice failure may result in the upper column moving relatively to the lower column. This disconnection 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, which 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 illustrated in Figure 1.

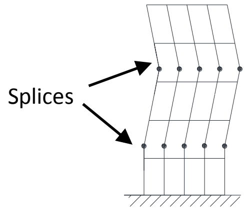

Figure 1. Possible partial sway mechanism of a frame due to low column splice stiffness/strength and splices at the same level

Splice rotation is more likely to occur in bolted rather than welded splices, but this effect has been ignored in research to date; Popov et al. [4], Bruneau et al. [5], Shen et al. [6], Shen et al. [7], Akbas et al. [8]. In this study, the effects of splice stiffness and location on moment and shear 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 drift ratios and displacements?

iii. How sensitive are demands to the location of splices?

iv. What is the likelihood of splice yielding?

2 MODELLING

The structure is the 9-story steel moment frame from the SAC steel project, which represents a mid-rise frame. Forty ground motions from SAC steel project, named LA1 to LA40 are used in the analyses, representing DBE and MCE levels of shaking**.** There are 20 ground motions at each shaking level [9].

2.1 Frame Properties

Figure 2 illustrates details of the 2D steel moment frame model and the location of splices. Splices are modelled as rotational springs for which the locations vary in each case studied. 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. Simulations were carried out in OpenSees [10] and nonlinear behaviour of frame elements was modelled with fiber hinges for which the post elastic stiffness ratio was set at 0.33.

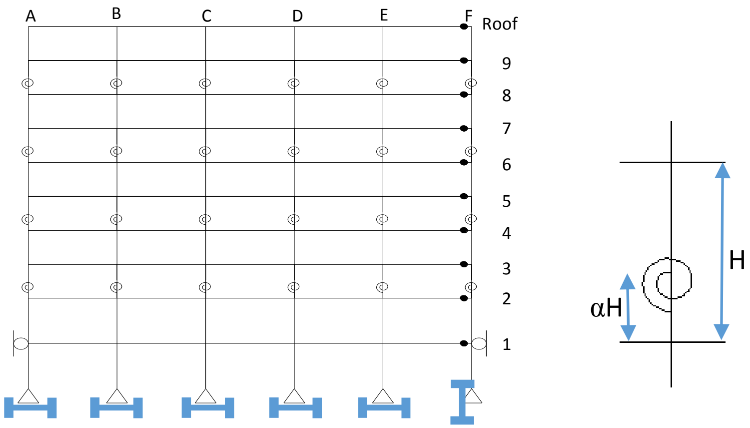

Figure 2. Details of 9 story 2-D model frame from the SAC project

Table 1. Details of 9 story frame elements [11]

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.

2.2 Splice Modelling

A parametric study was conducted to quantify splice stiffness and location effects on frame response. The splice yielding strength is assumed to be equal to 100% of plastic moment capacity of the smaller column at the location of splice. The location of splices is considered at one of 34, 50 or 66 percent of the story-height above the centre of the beam. The behaviour of the splices are modelled as elastic perfectly plastic and their stiffness is defined for different cases according to the diagram in Figure 3. The stiffness of splices selected is based on the limited literature available and the range was broadened for this study to also consider extreme flexible scenarios. Since the frame is indeterminate, the structure will be stable during the analyses even when splices have zero rotational stiffness.

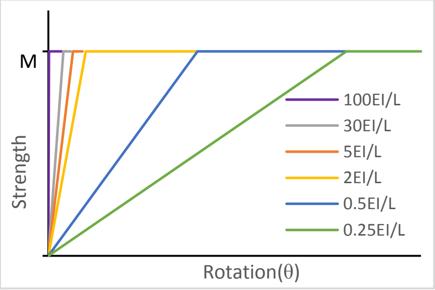

Figure 3. Flexural characteristics of splices as modelled for different cases examined

2.3 Earthquake Motions and Analyses

Two sets of twenty ground motion records, LA1-20 and LA21-40, were applied to the frame. These earthquake records represent DBE and MCE levels of ground motion with 10% and 2% possibility of occurrence in 50 years respectively. They are from the SAC suite of ground motions [9]. The responses of interest in this study are peak story drift ratio, peak story displacement, maximum moment and shear demand on splices. These values are reported as the median value for each suite of ground motions separately.

3 RESULTS

3.1 Time Period

The first and second mode periods of the frame for each case are presented in Table 2. The splice stiffness does not affect the period of the frame when splices are located in the middle of the columns. The first two mode periods are about 2% less than two other cases. These values are effectively the same and within construction variability. Overall, the period is decreasing with the increase in splice stiffness.

Table 2. First and second mode period of the frame with different splice stiffness and location

		Zero	0.25EI/L	0.5EI/L	2EI/L	5EI/L	30EI/L	100EI/L
α=0.34	First Mode	2.38	2.37	2.36	2.33	2.32	2.32	2.32
	Second Mode	0.91	0.90	0.89	0.88	0.87	0.87	0.87
α=0.5	First Mode	2.31	2.31	2.31	2.31	2.31	2.31	2.31
	Second Mode	0.87	0.87	0.87	0.87	0.87	0.87	0.87
α=0.66	First Mode	2.37	2.36	2.35	2.33	2.32	2.31	2.31
	Second Mode	0.89	0.88	0.88	0.87	0.87	0.87	0.87

3.2 Peak Drift Ratios

Figure 4 shows that when splices are in the middle of columns (α=0.50), the drift ratio is almost constant with the change in splice stiffness under DBE events. In addition, drift ratios are less than the other two cases (α=0.34, 0.66). Under MCE level events, in all three cases, splices with stiffness of zero, 0.25EI/L and 0.5EI/L cause larger drift ratios and show sensitivity to splice location. When splices are at two third of the column height, the drift concentration shifts to lower levels indicating significant change in demand and response due to splice location.

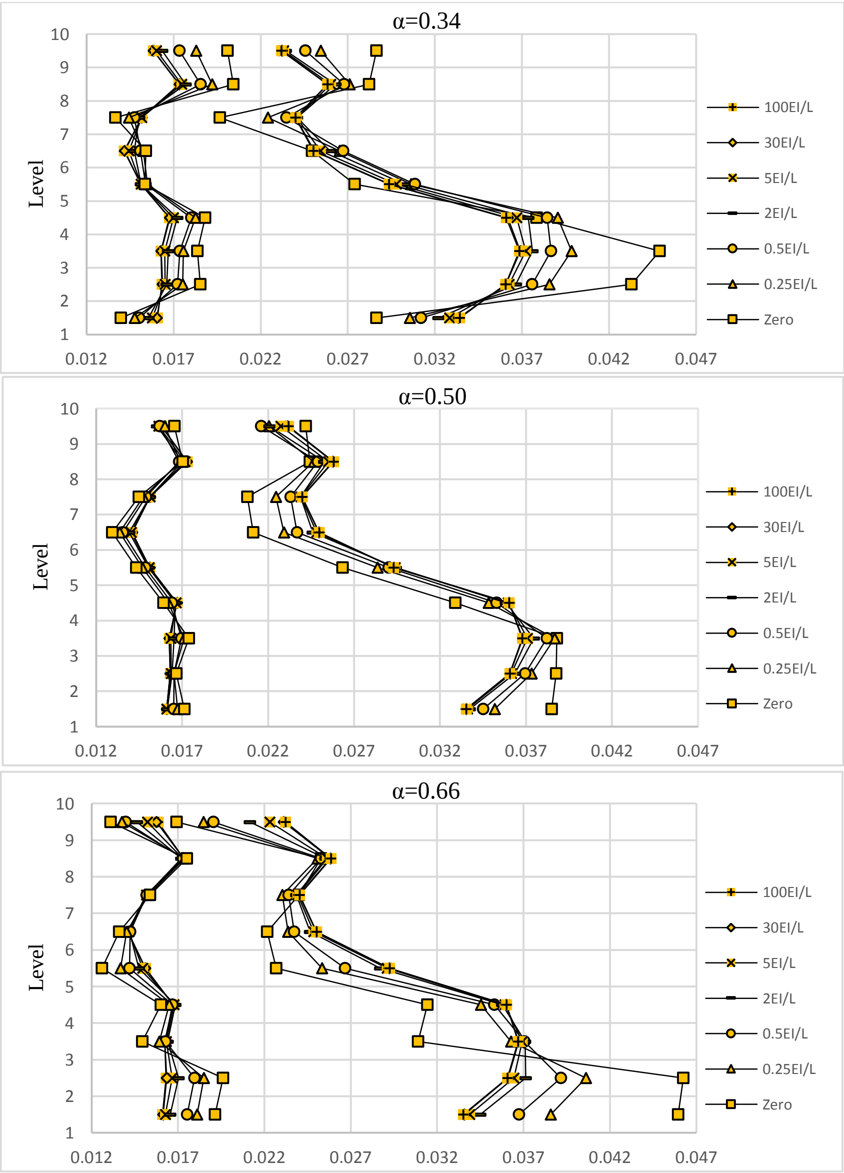

Figure 4. Frame median peak drift ratio profile for different splice stiffness legend and location (α=0.34, 0.50, 0.66) at DBE and MCE levels

3.2 Peak Story Displacements

Maximum displacement of the frame is not very sensitive to either stiffness or location of splices under DBE level events, as shown in Figure 5. However, in MCE level events, the stories below level 6 are more sensitive to splice stiffness when they are placed at the two third of columns. None of these results shows significant sensitivity of peak displacement to splice design factors.

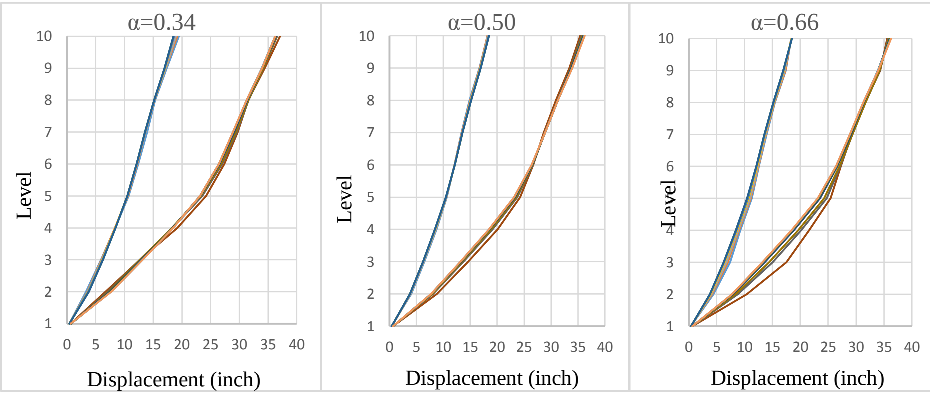

Figure 5. Frame median peak displacement profile for different splice stiffness and location (α=0.34, 0.50, 0.66) at DBE and MCE levels

3.4 Moment Demand on Splices

Figure 6 shows the median moment demand ratio of splices in the frame. The vertical axis is the ratio of the moment demand to the plastic capacity of the smaller section of the splice about which the column axis is oriented. Each line represents the median of the demand ratio variation with stiffness for each splice in the model. The dashed lines show the boundaries of the demand under DBE events and the solid black lines show the boundaries of the demand under MCE events. It is observed that the demand increases with splice stiffness. It can be also concluded that splices with stiffnesses larger than 30EI/L behave like a rigid connection. When splices are located in the middle of columns (α=0.50), the moment demand ratio is the least and about 16% and 27% in DBE and MCE events respectively. The maximum demand ratio of 47% occurs in MCE events, when splices are placed at two thirds the height of the column (α=0.66).

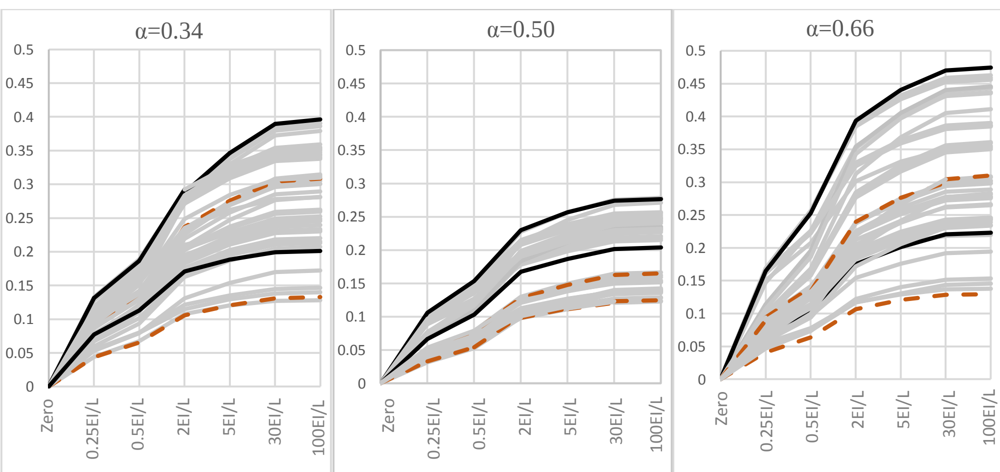

Figure 6. Median of maximum plastic moment ratio (M/M_P) of splices at DBE and MCE levels

3.5 Shear Demand on Splices

Figure 7 shows shear demand on column splices for different splice location. The vertical axis in the figure is the ratio of shear force to the shear capacity of smaller column section ($0.6f_ydt_w$). Each line represents the median of the demand ratio variation with stiffness for each splice in the model. The dashed lines show the boundaries of the demand under DBE events and the solid black lines show the boundaries of the demand under MCE events. It is seen that the demand is almost steady with different splice stiffnesses. In addition, the demand is not very sensitive to the splice location within the middle third of column. While the shear demand on splices in column F is very low, the value is very large in other splices of the frame which can reach up to about 48% of the section capacity. This is above the minimum capacity prescribed for splices in NZS3404.

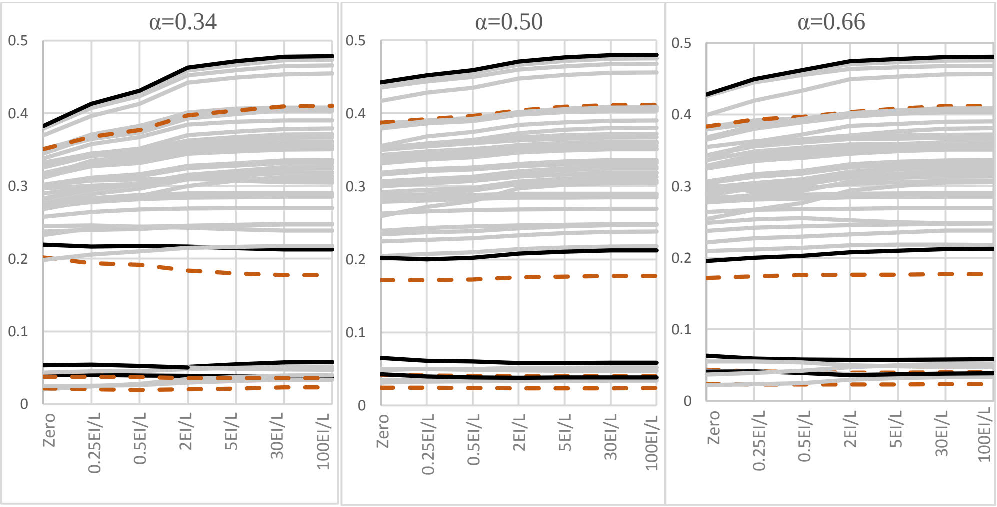

Figure 7. Median of maximum shear ratio of splices at DBE and MCE levels

4 CONCLUSION

Nonlinear time history analyses for a mid-rise steel moment frame have been conducted using DBE and MCE levels of ground motion. Column splices have been modelled as flexural springs characterised by strength and stiffness. Nonlinear numerical analysis of the effect of these properties and splice location on the responses of the frame clearly show that:

1- Flexible splices increased the frame first and second mode period by about 2 and 4 percent respectively when splices are located at either one or two third of column. Splice flexibility had no effect on the frame period when splices are in the middle of columns.
2- Peak frame displacements were not significantly affected by splice flexibility other than zero. But it increased the peak frame drift ratios by up to about 7% and 11% for non-zero splices when splices are at one third and two third of the column respectively.
3- It was observed that splices located in the middle of column have the least impact on the drift ratio and moment demand while splices located at two third of columns experienced larger demands.
4- Splice shear demands were up to about 41% and 48% of the nominal shear capacity under DBE and MCE shakings which is above the minimum requirement for splices according to NZS3404. The median of splice moment demands did not exceed 30% and 47% of the capacity for DBE and MCE shakings. However, there were splices with demands above 50% of their capacity in

MCE level ground motions. Future study is needed to assess the likelihood of any collapse or mechanism when splices are designed for shear and bending according to NZS3404.

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