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IMPROVING THE DYNAMIC RESPONSE OF STEEL STRUCTURES WITH PERIMETER MOMENT RESISTING FRAMES BY UTILIZING THE LATERAL RESISTANCE OF INTERIOR FRAMES

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SUMMARY

Survival of structural steel buildings after the 1994 Northridge earthquake, despite brittle failures observed at welded connections, and promising findings from studies on the dynamic response of frames with bolted connections constitute the motivation for this study. The primary objective is to evaluate the viability of improving the dynamic response of structural steel buildings with perimeter moment resisting frames (MRF) by considering the lateral resistance of interior frames as well as perimeter frames in the design of the structure. The scope of this paper, however, is limited to quantifying the lateral resistance provided by the interior frames and determining the influence of variations in the connection behavior on the dynamic response of three-story steel structures typically built in Boston and Los Angeles. A parametric study was conducted accordingly. Parameters considered in the study are: the initial stiffness and yield moment of interior-frame connections. It was concluded that the contribution from the interior frames in resisting ground excitations is significant in low-rise steel structures with comparable member sizes in the perimeter and interior frames, as in the case of Boston buildings. Furthermore, stiffening the interior-frame connections by an angle bolted to the bottom flange of the beam and to the flange of the column improved the dynamic response of the structure efficiently. For Los Angeles buildings, such retrofitting schemes were not observed to improve the dynamic response efficiently. Instead, design alternatives eliminating drastic stiffness differences between the perimeter and interior frame members may be more practical solutions for Los Angeles buildings. Further studies shall verify this hypothesis.

INTRODUCTION

The 1994 Northridge earthquake (Mw=6.7, USGS) was the first event to raise serious concerns about the insufficient ductility provided by structural steel MRF with welded-flange and bolted-web connections. Despite no record of collapsed structural steel buildings and no signs of yielding observed at investigated beams and columns, brittle joint failures associated with fracture of welds at beam bottom flanges were reported to be extensive after the 1994 Northridge event. Of the 167 buildings surveyed, 89 buildings

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(53%) were reported to have fractures at beam-column joints (FEMA 355E [1]). In one out of three buildings with connection damage, fractures were detected by visual inspection at more than 20 percent of the welded-flange bolted-web connections, referred to as welded connections hereafter. The 1995 Kobe earthquake (Mw=6.9, USGS) was the second event to emphasize the inadequate performance of MRF with welded connections in resisting ground excitations. The structural system tested by the event was a space MRF with welded connections at all joints and the extent of damage associated with weld fractures was even more severe. Based on the observations from these two events, research in U.S was mainly focused on improving the dynamic response of existing steel structures with perimeter MRF and suggesting design alternatives for the next generation of steel structures.

This study is focused on investigating the viability of improving the dynamic response of structural steel buildings with perimeter MRF by considering the lateral resistance of interior frames as well as that of perimeter frames in the design of the structure. Observations from the initial phase of this study, on the lateral resistance provided by interior frames and the influence of variations in the connection behavior on the response history of three-story steel structures typically built in Boston and Los Angeles, will be discussed in this paper. A simple retrofit involving the interior-frame connections will also be suggested.

MOTIVE AND OBJECTIVE OF THE STUDY

Since the 1980s, most steel buildings in California have been designed assuming that the lateral resistance of the structure is provided by the perimeter moment resisting frames, whereas the interior frames are designed to carry gravity loads only (Foutch and Yun [2]).The design typically results in large beam and column sections in the perimeter frames and relatively smaller member sections in the interior frames. Figures 1a and 1b show the details of typical bolted interior-frame connections and welded connections of the perimeter frame joints. In the earthquake-resistant design of steel structures with perimeter MRF, interior-frame connections are generally considered as nominally pinned and welded connections are, in theory, designed to provide the necessary ductility to resist ground excitations. However, brittle weld fractures observed after the 1994 Northridge event emphasized the limited ductility provided by welded connections, which, indeed, was not a new phenomenon (Popov and Pinkney [3], Popov et al. [4], Popov and Tsai [5], and Husain and Engelhardt [6]). Strictly, the survival of steel buildings with fractured welded connections revealed the reserve of ductility provided by bolted interior-frame connections, especially beyond deformation levels corresponding to fracture of joint welds.

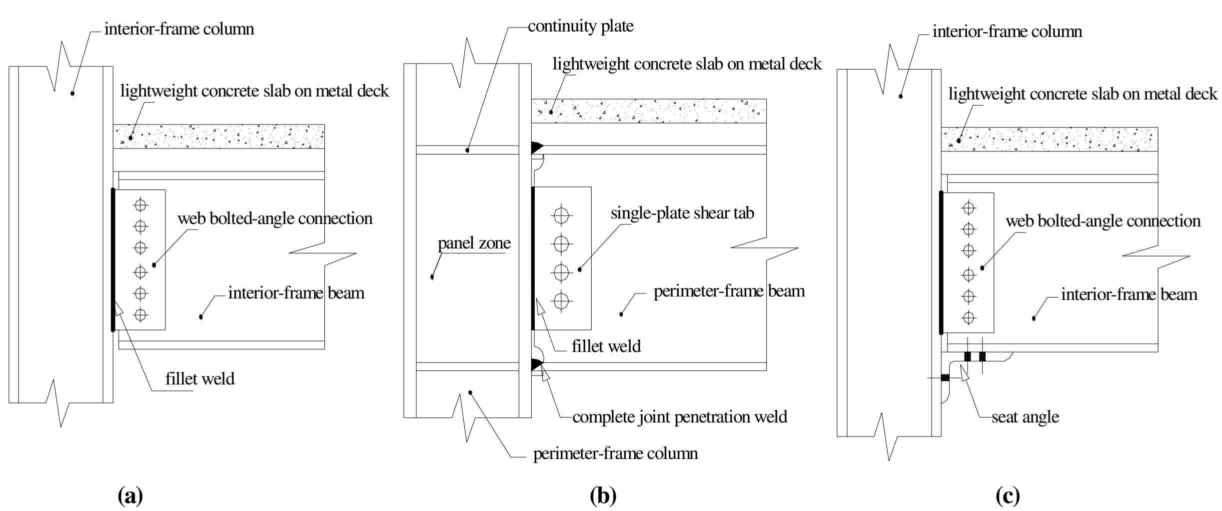


Figure 1. Types of connections considered in the study: (a) interior-frame bolted connection, (b) perimeter-frame welded connection, (c) retrofitted interior-frame connection

Observations from a detailed study on the response of typical bolted interior-frame connections under cyclic loading by Liu and Astaneh [7] verified that simple bolted connections (Fig.1a) have rotational stiffness and moment capacity. Previous research has also been focused on the possibility of using frames with bolted connections as part of the lateral load resisting system. Nader and Astaneh [8] conducted shaking table tests on one-story steel structures with simple bolted connections, partially restrained connections, and rigid connections. They suggested that if bolted connections are designed to participate in the nonlinear response they may enhance the dynamic performance of low-rise and mid-rise steel structures. Furthermore, Dubina et al. [9] observed that using bolted connections together with welded connections in frames may lead to improved hysteretic response. De Matteis et al. [10] had similar observations on the hysteretic response of steel structures comprising frames with welded connections and those with bolted connections. Shen and Akbas [11] conducted parametric studies on the seismic response of structures with perimeter frames and interior frames, and observed significant contribution from interior frames with bolted connections in resisting strong ground motions.

Survival of structural steel buildings after the 1994 event, despite brittle failures observed at welded connections, and promising observations from previous studies on the hysteretic response of frames with bolted connections constitute the motivation for this study. The primary objective of the study is to determine plausible design alternatives to improve the dynamic response structural steel buildings with perimeter MRF in different seismic risk zones by utilizing the lateral load resistance capacity of interior frames. The scope of this paper, however, is limited to quantifying the contribution from interior frames in resisting ground excitations and investigating an efficient retrofitting scheme involved with stiffening of the interior-frame bolted connections in three-story Boston and Los Angeles buildings.

DESCRIPTION OF THE ANALYSES

Response history analyses were conducted using a nonlinear analysis program, DRAIN-2DX [12], in order to investigate the influence of changes in the interior-frame connection behavior on the dynamic response of the structure. Ground acceleration histories recorded during the 1979 Imperial Valley, the 1989 Loma Prieta, the 1994 Northridge, the 1995 Kobe, and the 1999 Duzce earthquakes were considered in the analyses. The effective peak ground accelerations (PGA), velocities (PGV), and displacements (PGD) for the ground motions considered are provided in Table 1. For the analysis of three-story Boston buildings, the acceleration records were modified to have an effective PGA of 0.2g in order to consider credible levels of demand on a structure designed for Zone 2A of the Uniform Building Code [13].

Table 1. Characteristic values for the ground motions considered in the analyses

EVENT	STATION	DATE	PGA (g)	PGV (in./sec)	PGD (in.)
Imperial Valley	5054 Bonds Corner	15 Oct 1979	0.78	18	6.0
Loma Prieta	16 LGPC	18 Oct 1989	0.56	37	16.0
Northridge	24279 Newhall Fire Station	17 Jan 1994	0.59	38	15.0
Kobe	Takarazuka	16 Jan 1995	0.69	27	6.7
Duzce	BOLU090	12 Nov 1999	0.80	24	5.5

The analyses were focused on structural steel buildings representative of three-story Los Angeles and Boston designs. The structural layout and member sizes are consistent with those of typical steel buildings designed for zones of high and low seismic risk as part of the SAC Steel Project (MacRae [14]). Plan views and elevations of the buildings considered in the analyses are presented in Figure 2. Perimeter MRF member sizes are also included in the figure. Beams used in the interior frames are of W24x55 sections, and columns are of W14x90 sections.

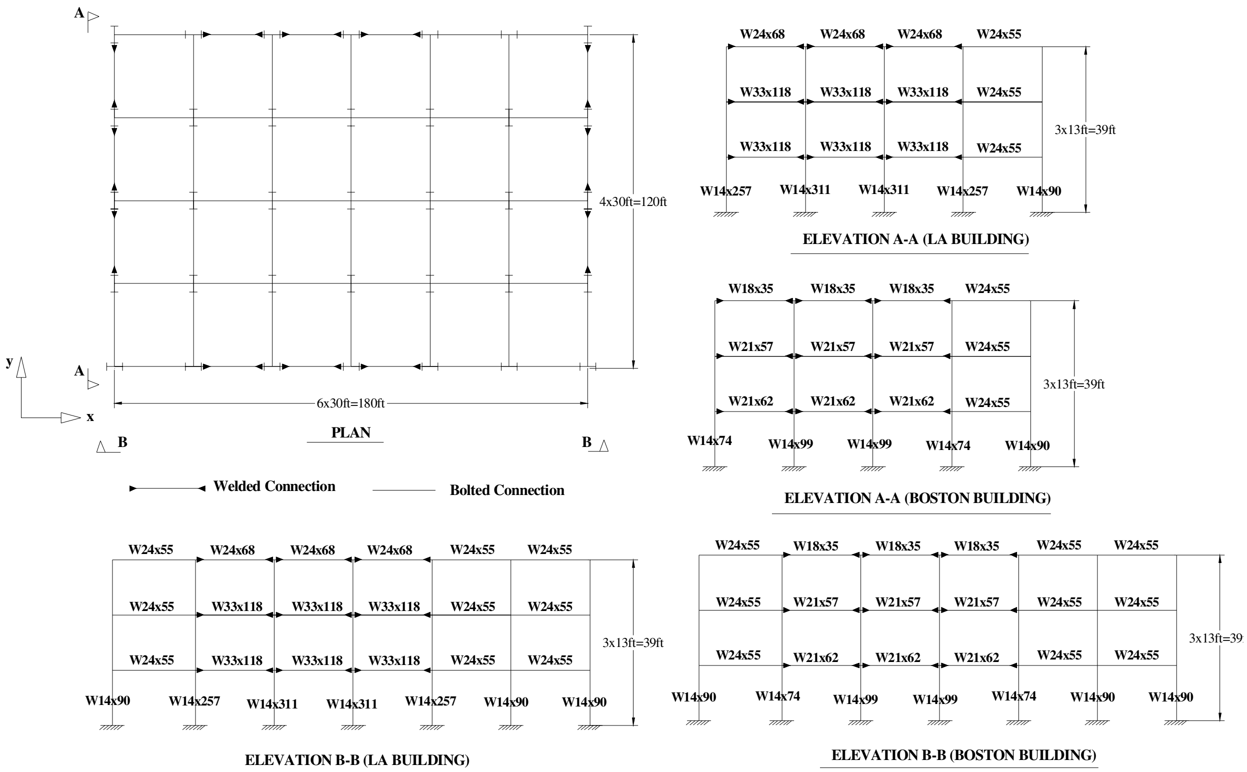


Figure 2. Plan view and elevations for typical three-story Los Angeles and Boston steel buildings

Dynamic response of the structures was studied in the y-direction, which is the strong axis direction for interior-frame connections. The two-dimensional models considered in the analyses of the buildings comprise two perimeter frames and five interior frames. Columns were assumed to be fixed at their bases. All beams and columns were assumed to remain linear elastic throughout the response. It is important to note that, this assumption is consistent with the observations from surveys of steel buildings after the 1994 Northridge earthquake (FEMA 355E [1]). All connections were modeled to have bilinear moment-rotation response with equal loading and unloading stiffnesses. Limiting the nonlinearity to the connections also enables to monitor the changes in the dynamic response of the structure corresponding to changes only in the behavior of the connections.

Any retrofit suggested for the interior-frame connections would change the stiffness and the strength of the connection. Therefore, connection initial stiffness and yield moment were selected as the main parameters of the study. In order to investigate the influence of variations in the connection stiffness and strength individually, it was decided to vary one parameter at a time. The values considered for relevant parameters were modified while changes in the maximum base shear forces resisted by interior frames and perimeter frames, maximum interstory drift ratios, and fundamental period of vibration were monitored for the structures. The maximum base shear forces resisted by interior and perimeter frames were considered as a measure of the lateral resistance provided by relevant frames. Furthermore, maximum interstory drift ratio was observed to be the most critical drift value for evaluating the performance of the structure in resisting ground motions. The range of values considered for the variables are summarized below.

Initial stiffness values considered for the interior-frame connections in the analyses range from zero, representing a nominally pinned connection, to 20 times the flexural rigidity ($(EI/L)_{beam}$) of the adjoining beam, where E is the elastic modulus, I is the moment of inertia for the beam, and L is the centerline to centerline distance between columns. A connection stiffness of $20(EI/L)_{beam}$ was considered to represent that of a welded connection, which is within the range of stiffnesses suggested for welded connections by Leon et al. [15]. Among the connection stiffnesses considered in the analyses, $2.5(EI/L)_{beam}$ corresponds to the stiffness of the simple bolted connection shown in Fig.1a, whereas $6(EI/L)_{beam}$ corresponds to the initial stiffness of the simple bolted connection retrofitted with a seat angle as shown in Fig.1c. These values are consistent the measurements reported by Liu and Astaneh [7]. It is important to note that the connection model used in DRAIN-2DX requires defining the yield moment and stiffness beyond yield for the connection considered in addition to the initial stiffness. The connection yield moment was considered as 0.15 times the yield moment of the beam (My_{beam}) when the slab is in tension and $0.35My_{beam}$ when the slab is in compression. The connection stiffness beyond yield was assumed to be one-half of the initial stiffness; initial stiffness being the main parameter varied. For a connection stiffness of $2.5(EI/L)_{beam}$, the values selected for the yield moment and stiffness beyond yield were observed to represent the maximum energy dissipated by simple bolted connections at rotation levels corresponding to maximum interstory drift ratios on the order of two percent.

The influence of changes in the connection yield moment on the lateral resistance provided by interior frames and the maximum interstory drift ratios obtained for the structures was studied for a connection stiffness of $(6EI/L)_{beam}$. As mentioned previously, $(6EI/L)_{beam}$ corresponds to the stiffness of an interior-frame connection retrofitted using a seat angle (Fig.1c). Based on the measurements from the study conducted by Liu and Astaneh [7], the yield moment considered for this connection was varied between $0.3My_{beam}$ and $0.75My_{beam}$ for Grade 50 steel. The stiffness beyond yield was considered to be ten percent of the initial stiffness for the retrofitted interior-frame connection.

Furthermore, the stiffness of welded connections in the perimeter frames was assumed to be $20(EI/L)_{beam}$. The yield moment of welded connections was set equal to My_{beam} for Grade 50 steel. The response of the welded connections was assumed to be elastic perfectly plastic.

RESULTS OBTAINED FROM THE PARAMETRIC STUDY

Results obtained from the parametric study described above are summarized in this section. The indications of these observations will be discussed in the following section.

Influence of Initial Stiffness Considered for Interior-Frame Connections on the Maximum Base Shear Force Resisted by Interior Frames

The rotational stiffness of interior frame connections was varied between $0(EI/L)_{beam}$ and $20(EI/L)_{beam}$ and the corresponding changes in the maximum base shear force resisted by interior frames and perimeter moment resisting frames were investigated for three-story Los Angeles and Boston buildings. Figure 3 shows that, in the three-story Los Angeles building approximately one-third of the stiffness is from the interior frame columns (interior frame connections have a stiffness of $0(EI/L)_{beam}$). If the interior-frame connection stiffness is considered to be $2.5(EI/L)_{beam}$, approximately 40 percent of the overall stiffness can be attributed to the interior frames. In other words, the stiffness of an interior frame is approximately one-fourth of that of a perimeter frame. Interior frames were observed to provide 44 percent of the overall stiffness when the connection stiffness equals $6(EI/L)_{beam}$. The average increase in the lateral resistance provided by interior frames due to increase in the connection stiffness beyond $6(EI/L)_{beam}$ was observed to be six percent.

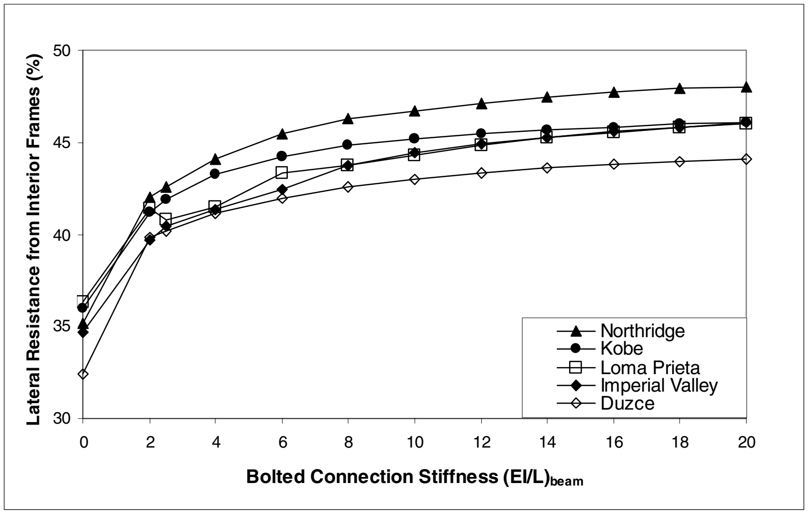


Figure 3. Influence of connection stiffness on the lateral resistance provided by interior frames in Los Angeles buildings

In the typical three-story Boston building, approximately 57 percent of the stiffness was observed to be from the interior frame columns (interior frame connections have a stiffness of $0(EI/L)_{beam}$). The contribution from the interior frame columns of the Boston building is approximately 70 percent more than that observed in the Los Angeles building. If the interior frame connections are considered to have a rotational stiffness of $2.5(EI/L)_{beam}$, it was observed that 65 percent of the stiffness is from the interior frames. In this case, an interior frame provides 75 percent of the lateral resistance provided by a perimeter frame (Fig. 4). For an interior frame connection stiffness of $20(EI/L)_{beam}$, the interior frames were observed to provide 70 percent of the lateral stiffness, for which the structural system responds similar to a space frame. The average increase in the lateral resistance provided by interior frames due to increase in the connection stiffness beyond $6(EI/L)_{beam}$ was observed to be three percent.

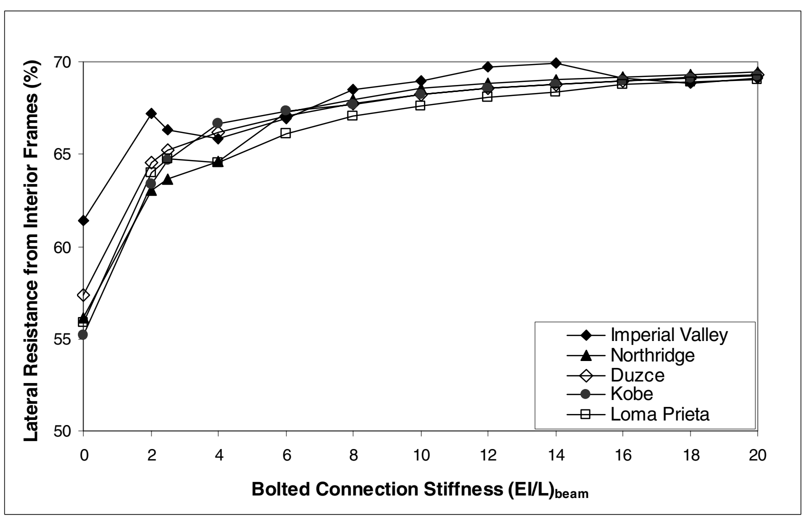


Figure 4. Influence of connection stiffness on the lateral resistance provided by interior frames in Boston buildings

Influence of Yield Moment Considered for Interior-Frame Connections on the Maximum Base Shear Force Resisted by Interior Frames

In the previous section, it was observed that connection stiffnesses beyond that of a typical interior-frame connection retrofitted by a seat angle (Fig.1c) leads to virtually no further increase in the lateral resistance provided by interior frames. Therefore, the influence of changes in the connection yield moment on the maximum base shear force resisted by interior frames was investigated only for a connection stiffness of $6(EI/L)_{beam}$. It was observed that varying the moment capacity of the connection between $0.3My_{beam}$ and $0.75My_{beam}$ resulted in negligible changes (up to seven percent increase) in the lateral resistance provided by the interior frames of three-story Los Angeles building (Figure 5). In the three-story Boston building, variations in the strength of interior frame connections were observed to result in no change in the lateral stiffness provided by interior frames.

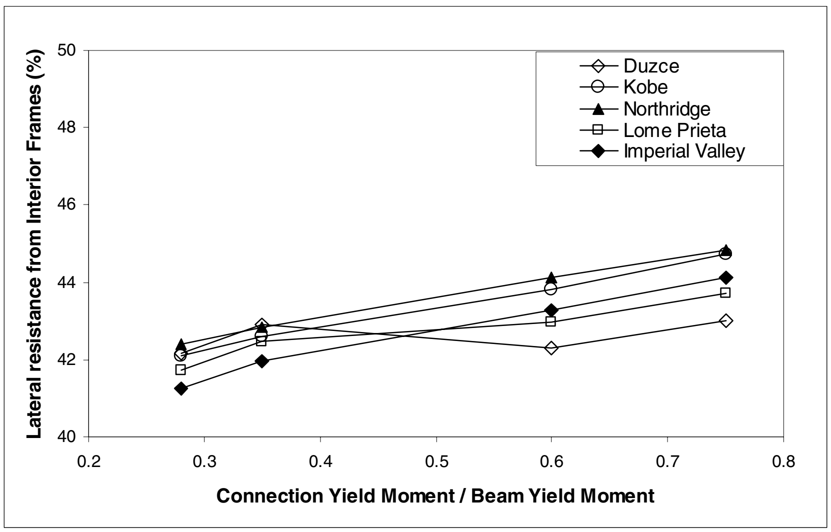
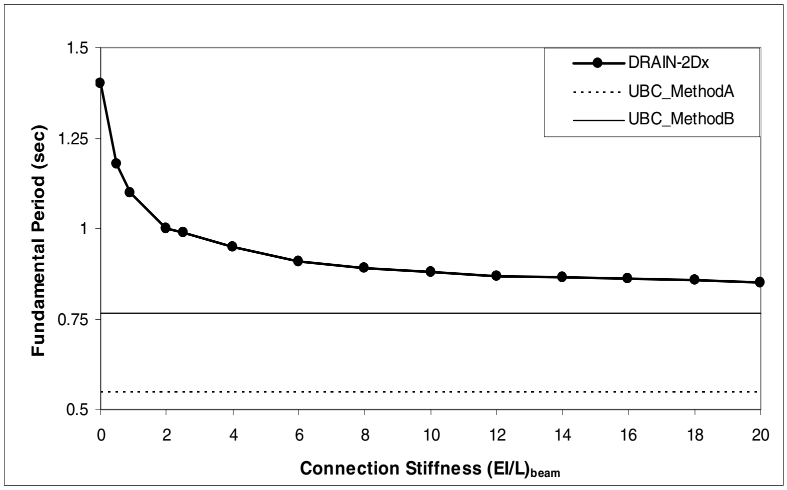


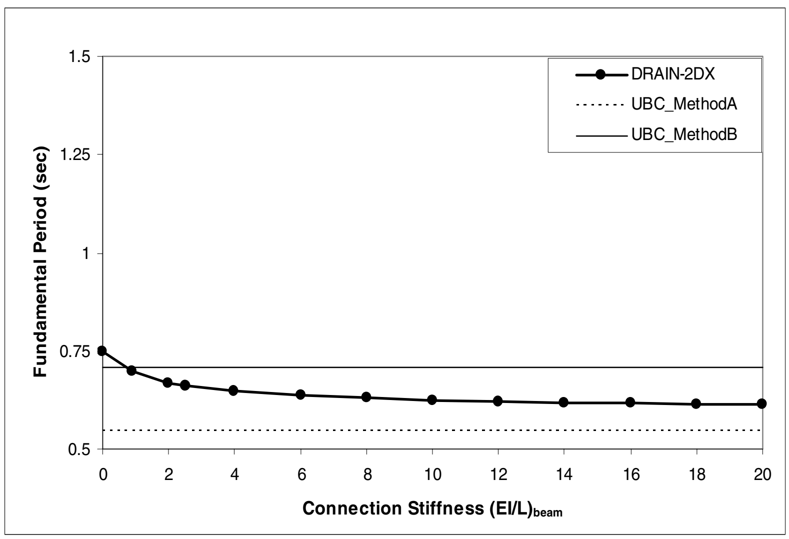
Figure 5. Influence of connection yield moment on the lateral resistance provided by interior frames in Los Angeles buildings

Influence of Initial Stiffness Considered for Interior-Frame Connections on the Estimates of Fundamental Period

The rotational stiffness of interior frame connections was varied between $0(EI/L)_{beam}$ and $20(EI/L)_{beam}$ and the corresponding changes in the fundamental period of vibration estimated for Boston and Los Angeles buildings were monitored. It was observed that ignoring the inherent rotational stiffness of interior-frame connections ($2.5(EI/L)_{beam}$) and assuming these connections to be nominally pinned leads to 40 percent increase in the fundamental period estimated for the Boston building. On the other hand, increasing the connection stiffness from $2.5(EI/L)_{beam}$ to $6(EI/L)_{beam}$ led to an eight percent reduction in the fundamental period computed. Increasing the connection stiffness from $6(EI/L)_{beam}$ to $20(EI/L)_{beam}$ resulted in a reduction of six percent in the period of the structure. In the three-story Los Angeles building, however, ignoring the stiffness of interior-frame connections was observed to result in a 14 percent longer fundamental period. The reduction in the period of the structure was observed to be three percent for an increase in the connection stiffness from $2.5(EI/L)_{beam}$ to $6(EI/L)_{beam}$ and four percent for an increase from $6(EI/L)_{beam}$ to $20(EI/L)_{beam}$. Figures 6a and 6b show the distribution of fundamental periods computed for Boston and Los Angeles buildings with stiffness considered for the interior-frame connections. The fundamental period estimates suggested by Method A and Method B in Section 1630.2.2 of the Uniform Building Code [13] are also included in the figures to give an idea on the order of periods used in the earthquake-resistant design of Boston and Los Angeles buildings. These figures emphasize the importance of considering the interior-frame connection stiffness in order to obtain credible estimates for the fundamental period. However, it is important to note that a stiffness beyond that of a fairly simple connection ($2.5(EI/L)_{beam}$) considered for the interior-frame connections leads to trivial further changes in the estimates of fundamental period.



(a)



(b)

Figure 6. Influence of connection stiffness on the estimates of fundamental periods obtained for: (a) Boston buildings, (b) Los Angeles buildings

Influence of Initial Stiffness Considered for Interior-Frame Connections on the Maximum Interstory Drift Ratio

The rotational stiffness of interior frame connections was varied between $0(EI/L)_{beam}$ and $20(EI/L)_{beam}$ in order to monitor the corresponding changes in the maximum interstory drift ratios obtained for Los Angeles and Boston buildings. Figure 7 shows that, in general, increasing the bolted connection stiffness leads to reduced maximum interstory drift ratios in three-story Los Angeles buildings. However, there are cases for which an increase in the connection stiffness corresponds to an increase in the drift ratios computed, whereas for three-story Boston buildings increasing the connection stiffness was observed to result in reduced maximum interstory drift ratios for most cases (Fig.8). Furthermore, from Figures 7 and 8 it can be observed that increasing the connection stiffness beyond $6(EI/L)_{beam}$ does not improve the displacement response any further. It is important to note that both Los Angeles and Boston buildings satisfy the drift requirements prescribed as in Section 16.30.10 of the Uniform Building Code [13].

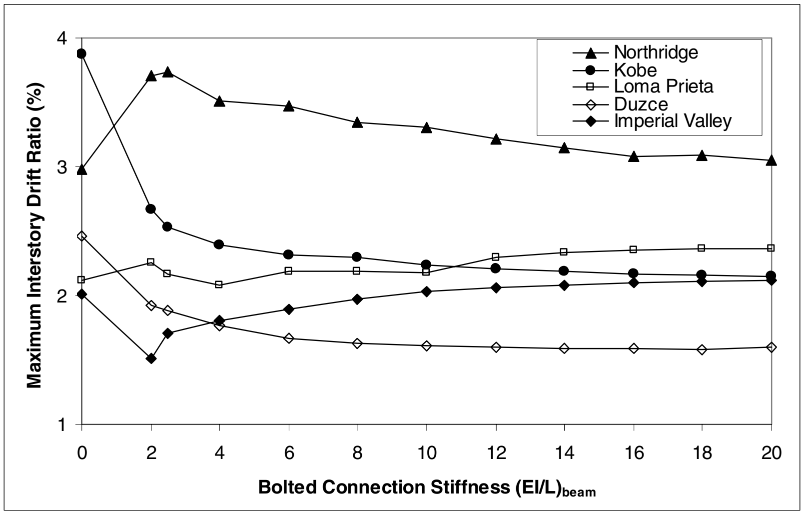


Figure 7. Influence of connection stiffness on the maximum interstory drift ratios for Los Angeles buildings

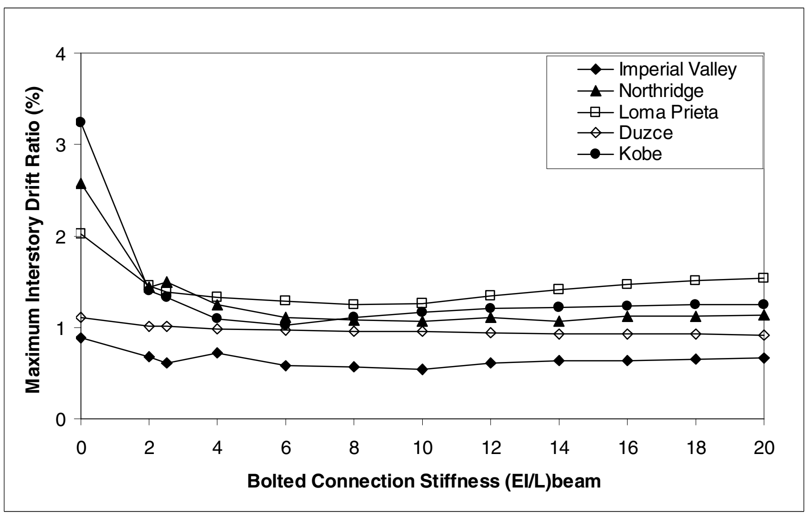


Figure 8. Influence of connection stiffness on the maximum interstory drift ratios for Boston buildings

It can be seen from **Figure 9** that ignoring the inherent rotational stiffness of interior frame connections led to 20 percent and 50 percent increases (positive y-values in the figure) in the maximum interstory drift ratios calculated for Los Angeles buildings, as well as decreases (negative y-values in the figure) up to 20 percent. The average change in the drift ratios for the ground motions considered is 16 percent reduction. For Boston buildings, however, it was observed that neglecting the stiffness of interior-frame connections may result in maximum interstory drift ratios up to 2.4 times larger than those expected to be observed in the displacement response of the structure. The average change in the drift ratios obtained for the ground motions considered is 64 percent reduction.

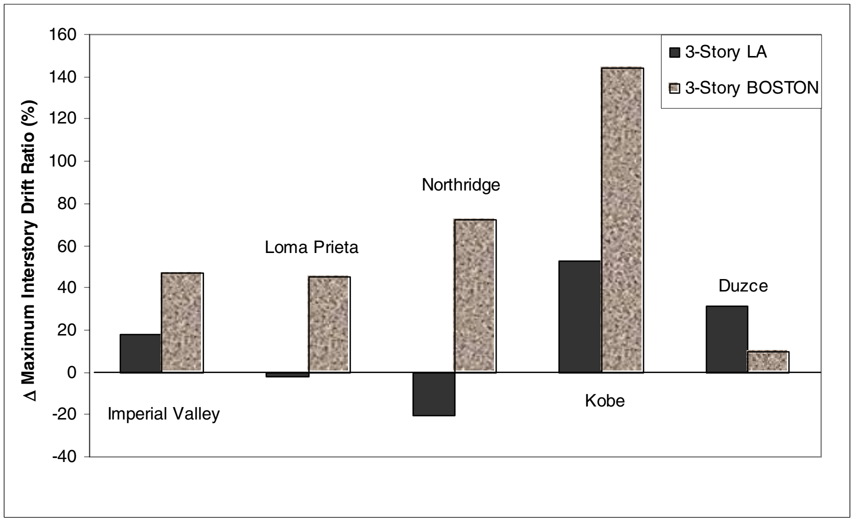


Figure 9. Changes in the maximum interstory drift ratio due neglecting the rotational stiffness of interior-frame connections

From **Figure 10** it can be observed that increasing the connection stiffness from $2.5(EI/L)_{beam}$ to $6(EI/L)_{beam}$ by adding a seat angle at the bottom flange of the beam is more efficient in reducing the maximum interstory drift ratios obtained for the Boston building than those obtained for the Los Angeles building. Reductions (negative y-values in the figure) in the maximum interstory drift ratios were observed to range from five percent to 25 percent for the Boston building, with an average of 13 percent. Reductions as well as increases up to ten percent were observed in the maximum interstory drift ratios computed for the Los Angeles building, with an average of three percent reduction.

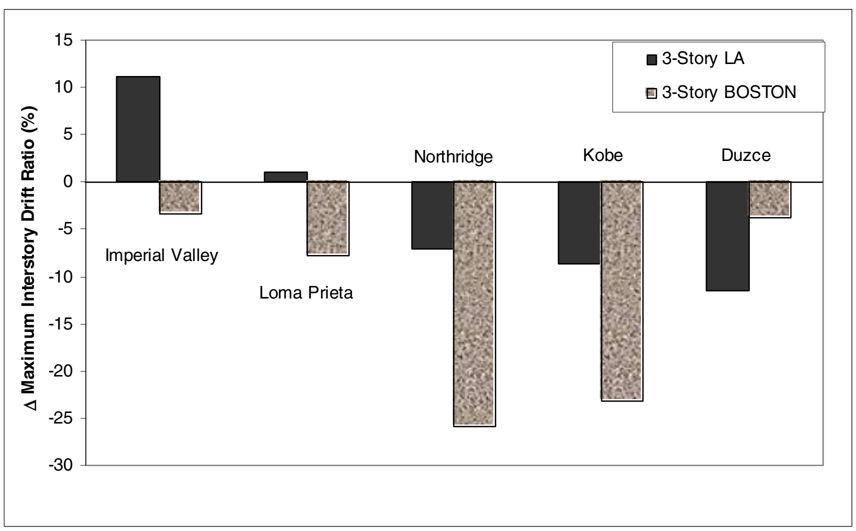


Figure 10. Changes in the maximum interstory drift ratio due to retrofitting the interior-frame joints with a seat angle at the beam bottom-flange

Furthermore, comparison of the information summarized in Figures 9 and 10 shows that the most drastic change in the maximum interstory drift ratio occurs when the inherent rotational stiffness of interior-frame connections is neglected especially in the Boston building.

Influence of Yield Moment Considered for Interior-Frame Connections on the Maximum Interstory Drift Ratio

The influence of variations in the yield moment considered for interior-frame connections on the maximum interstory drift ratio obtained for Los Angeles and Boston building was investigated. The connection stiffness considered was $6(EI/L)_{beam}$. It was observed that varying the connection yield moment between $0.3My_{beam}$ and $0.75My_{beam}$ for the interior-frame connections of the Los Angeles building leads to negligible (up to five percent) reductions in the maximum interstory drift ratios (Figure 11). Furthermore, variations in the yield moment considered for the interior-frame connections was not observed to have any influence on the maximum interstory drift ratios computed for the Boston building.

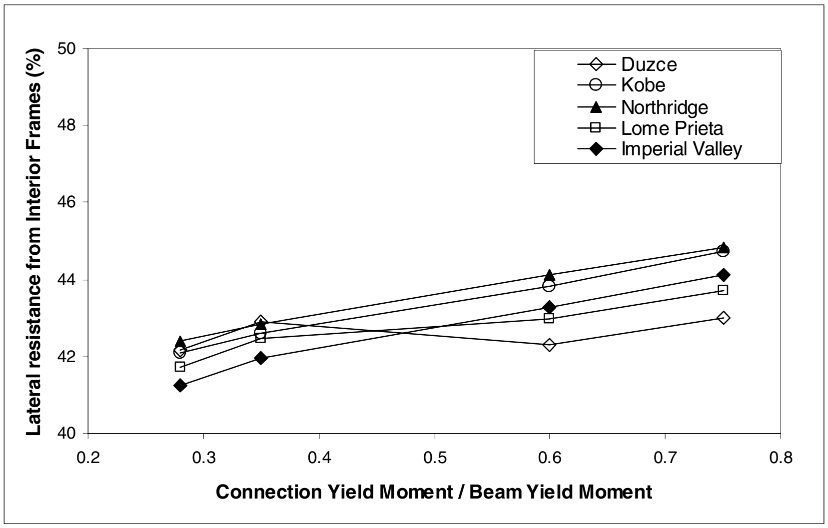


Figure 11. Influence of connection yield moment on the maximum interstory drift ratios for Los Angeles building

SUMMARY AND DISCUSSION OF THE RESULTS

Results obtained from the parametric study described above suggest that the contribution from the interior frames in resisting ground excitations is sensitive to the relative stiffnesses of perimeter frame and interior frame members. If simple bolted connections (Fig. 1a) are used in the interior frame joints, an interior frame was observed to resist 75 percent of the maximum lateral load resisted by a perimeter frame (Fig.4) when the member sizes in the perimeter frames and interior frames are comparable, such as in the case of three-story Boston buildings. Furthermore, it was observed that dynamic response of such steel structures can be improved by simply increasing the stiffness of interior-frame bolted connections, as shown in Fig.1c. On the other hand, for a structure with noticeable stiffness difference between interior frame and perimeter frame members, such as in the case of three-story Los Angeles buildings, an interior frame was observed to resist only 25 percent of the maximum lateral load resisted by a perimeter frame (Fig.3). In this case, increasing the stiffness of interior-frame connections led to negligible increases or decreases in the displacement response history of the structure (Fig.10). Furthermore, lateral resistance provided by the interior frames was observed to virtually insensitive to the frequency content of the ground motions when the interior and perimeter frames have comparable member sizes (Fig.4).

It was concluded that recognizing the lateral stiffness provided by the interior frames is crucial in obtaining credible estimates for the fundamental period of structures especially in Boston-type buildings (Figs 6a and 6b). Observations on the estimates of fundamental period also verify the significant lateral resistance provided by the interior frames in Boston buildings. The fundamental period estimated for the

Boston building was observed to be 40 percent longer when the inherent rotational stiffness of interior-frame connections was ignored (Fig. 6a). The longer period resulted in up to 2.4 times larger maximum interstory drift ratios (Fig.9). Furthermore, increasing the connection stiffness from $2.5(EI/L)_{beam}$ to $6(EI/L)_{beam}$ led to eight percent reduction in the period estimate, which resulted in up to 25 percent reductions in the maximum interstory drift ratios (Fig.10). Because the interior frames provide approximately 75 percent of the overall stiffness, increasing the stiffness of interior-frame connections was observed to result in reduced interstory drift ratios for the ground motions considered.

In the Los Angeles building, however, neglecting the rotational stiffness of bolted connections resulted in 14 percent increase in the fundamental period of the structure, which led to both increases and decreases in the displacement response of the structure. Furthermore, increasing the connection stiffness from $2.5(EI/L)_{beam}$ to $6(EI/L)_{beam}$ led to three percent reduction in the period estimate, which resulted in up to 10 percent increase and decrease in the maximum interstory drift ratios (Fig.10). Because the stiffness of the perimeter frames was observed to dominate the response, increasing the stiffness of bolted connections from $2.5(EI/L)_{beam}$ to $6(EI/L)_{beam}$ resulted in negligible changes in the maximum interstory drift ratios obtained for the structure (Fig.10).

The yield moment considered for the retrofitted interior-frame connection was observed to have trivial influence, if any, on the lateral resistance provided by the interior frames as well as the maximum interstory drift ratios computed for the structures. Therefore, increasing the stiffness of the interior-frame connections seems to be the main concern for retrofitting purposes. However, it is important to note that, increasing the connection stiffness beyond that of a simple bolted connection retrofitted by a seat angle at the bottom flange of the beam ($6(EI/L)_{beam}$) does not lead to any further improvement in the displacement response of the structure (Figs. 7 and 8).

CONCLUSIONS

Based on the findings from the preliminary parametric study discussed in this paper, it was concluded that the dynamic response of low-rise steel structures with comparable member sizes in the perimeter frames and interior frames is more consistent and plausible. Lateral resistance provided by the interior frames should be recognized in the design of such structures. Moreover, it is possible to improve the dynamic response of structures with comparable member sizes in the perimeter and interior frames by simply increasing the stiffness of interior-frame connections. However, increasing the stiffness of interior-frame connections beyond that of a simple bolted-connection retrofitted by an angle bolted to the bottom flange of the beam and to the flange of the column does not result in additional improvement in the displacement response of the structure. There is need for further studies in order to verify that the proposed retrofitting scheme is suitable for mid-rise Boston buildings as well.

It was decided that problems concerning the seismic response of Los Angeles buildings cannot be resolved efficiently by increasing the stiffness of interior-frame connections because the response is dominated by the robust perimeter frames existing in the structure. Instead, designing the future generation of Los Angeles steel buildings in such a way that the perimeter frames and interior frames have comparable member sizes, as in the case of Boston buildings, seems to be a plausible design alternative. This design alternative is expected to enable utilizing the reserve of lateral load resistance capacity provided by the interior frames. However, the credibility of improving the performance of low-rise and mid-rise Los Angeles buildings in resisting ground motions thorough such a design approach needs to be verified by further studies.

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