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SEISMIC PERFORMANCE OF OLDER PR FRAMES IN AREAS OF INFREQUENT SEISMICITY

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SUMMARY

This study evaluates and compares the seismic performance of older steel moment frames with partially restrained (PR) connections. Typical 4-, 6-, and 8-story older frames were designed based on the 1952 AISC code using the "Wind Moment Frame" method. Nonlinear time-history analyses were conducted for thirty Mid-America ground motions using a modified version of the DRAIN-2DX program. The primary performance indices used were connection rotations, interstory drift angles (ISDA), panel zone deformations, and plastic hinge rotations. The results show that these frames may be susceptible to serious damage if subjected to a large earthquake (2% in 50 years) if the frames were designed for wind forces only. However, the existing structures would perform reasonably well for a moderate event (10% in 50 years).

INTRODUCTION

Since their earliest versions, American steel design codes have permitted the use of simplified procedures for the design of moment frames subjected to gravity and wind loads. These procedures are commonly labeled as Type 2 construction or the "Wind Moment Frame" (WMF) method. Figure 1 shows a schematic illustration of the basic concept behind the WMF method. This method states that the beams can be designed for the gravity loads based on assuming pinned connections at the beam ends as shown in Figure 1(a), and that the joints and members can be designed for the wind forces assuming fixed connections as shown in Figure 1(b). The actual behavior of the structural system, which is often that of a partial strength (PS), partially restrained (PR) system, can be obtained by superposing these two cases, as shown in Figure 1(c).

The assumption made for the gravity load design shown in Figure 1.1(a) can be justified on the basis of the shakedown theorems of plastic analysis [1]. These theorems state that if the connection strength degrades with cycling, as would occur with repeated wind loads, a safe condition for design is given by the assumption of pinned ends. The assumption that the connections are rigid for lateral loads simplified the design, as the portal, cantilever or similar methods could be applied. These methods assume rigid

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connections and simplified distributions of the inflection points, columns shears under lateral loads, or similar assumptions that results in a statically determinate problem amenable to hand calculations.

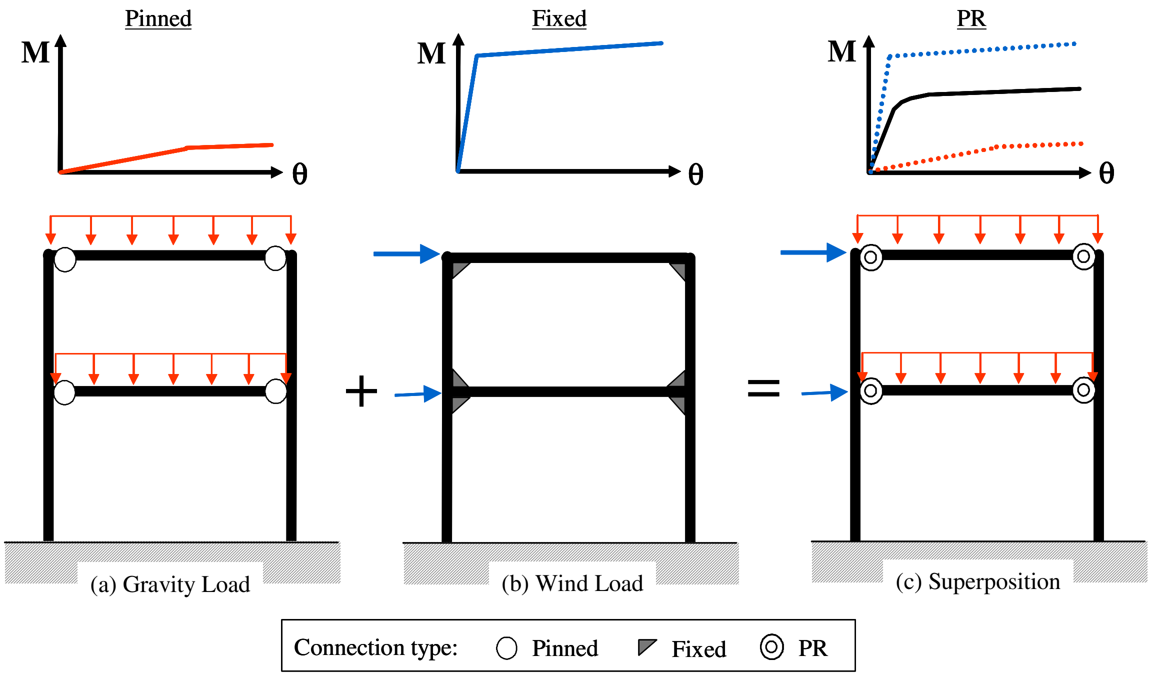


Figure 1 - Schematic View of the Wind Moment Frame Method

In the original development of the method, the moments induced by wind were assumed to be relatively small when compared to the plastic moment capacity of the beams. In general, the wind forces were based on assuming a uniform pressure of 20 psf (956Pa) on the projected area of the building. Since the connecting elements were designed only for the wind loads and the preferred connection method was by bolting, angles or small T-stub sections were often used to carry the forces between the beam and the column flanges. Because of this design approach, the connections, in general, were weaker and more flexible than the beams and columns, resulting in what we label today as a partial strength (PS), partial restraint (PR) system.

The WMF method had obvious advantages in the era before micro-computers made the analysis for more complex connection conditions and load combinations possible. In the WMF method, the design requirement for the connections is strictly a strength one based on the wind forces obtained from the analysis. No specific checks are made for the stiffness of the connection or its effect on the distribution of forces and resulting displacements. Many studies have shown that the method is reasonable for regular frames with strong PR connections [2] subjected to moderate lateral loads. However, the method, as currently allowed in the steel design specification, does not have any limitations on its applicability insofar as height of the structure or irregularities in plan, stiffness or strength distribution.

The WMF method has been and continues to be widely utilized in the Central and Eastern U.S., resulting in a very large population of structures having been designed without a rigorous analysis with respect to lateral loads. As the seismic risk in Mid-America has been considerably increased in recent design codes, at least two practical problems need to be addressed. First, numerous moment-resisting frames designed by this method in these areas have been used for critical facilities, mostly hospitals. The vulnerability of these and similar critical facilities needs to be clearly understood. Second, older structures that are being rehabilitated are required to meet the most recent code when more than 5% of the building is renovated. In these cases, a good estimate of the actual strength of these structures will be of great practical and economic significance, as it will impact what type and how much strengthening or stiffening of the primary structural system will be required. These and similar issues related to understanding the seismic behavior of PR/PS structural systems are the motivation behind this research.

PROTOTYPE FRAME

Three sets of prototype PR frames – Older, Intermediate, and Modern steel moment frames – were designed. For each of these prototypes, 4-, 6- and 8-story configurations were detailed. In addition, the frames were designed for 4 separate cities (Memphis (TN), St. Louis (MO), Charleston (SC), and Carbondale (IL)), representing a spectrum of seismic demands for the Eastern USA.

The Older frames represent steel buildings designed in the period 1940-1960. These frames were designed in accordance with 1952 AISC Steel Construction manual [3] for gravity loads and the 1948 Joint Committee Recommendations of San Francisco [4] for lateral forces. The Intermediate frames represent buildings designed in the period 1970-1994 and were designed in accordance with 1989 AISC ASD [5] for gravity loads and 1991 UBC [6] for lateral forces. These frames are representative of a pre-Northridge steel frame. Buildings designed between 1997 and today are represented by the Modern frames, which were designed in accordance with 1998 AISC LRFD [7] and 2000 IBC [8] for gravity loads and lateral forces, respectively. The Older frames were designed following the WMF method and will be the only type of frame discussed in this paper due to space limitations.

The Older frames are 3 bays by 3 bays, with bay lengths of 20ft-10ft-20ft (6.1m-3.05m-6.1m) in the North-South (NS) direction and three 20ft (6.1m) in the East-West (EW) direction. The building plan and an elevation of the analytical model are shown in Figures 2 and 3 for a typical Older 8-story structure. Typical member sizes are given in Table 1, and the design spectra used are shown in Figure 4. Note that different soil types were used at some locations to infer the sensitivity of the designs to this parameter.

Table 1 – Member sizes for typical 8-story Older frame

Story	**Column**		**Beam**	
	Exterior	**Interior**	**Exterior**	**Center**
8	W10x33	W10x39	W16x50	W8x21
7	do	do	W18x55	W14x34
6	W10x45	W10x60	do	do
5	do	do	do	do
4	W10x68	W10x77	do	do
3	do	do	do	do
2	W10x88	W10x112	do	do
1	do	do	do	do

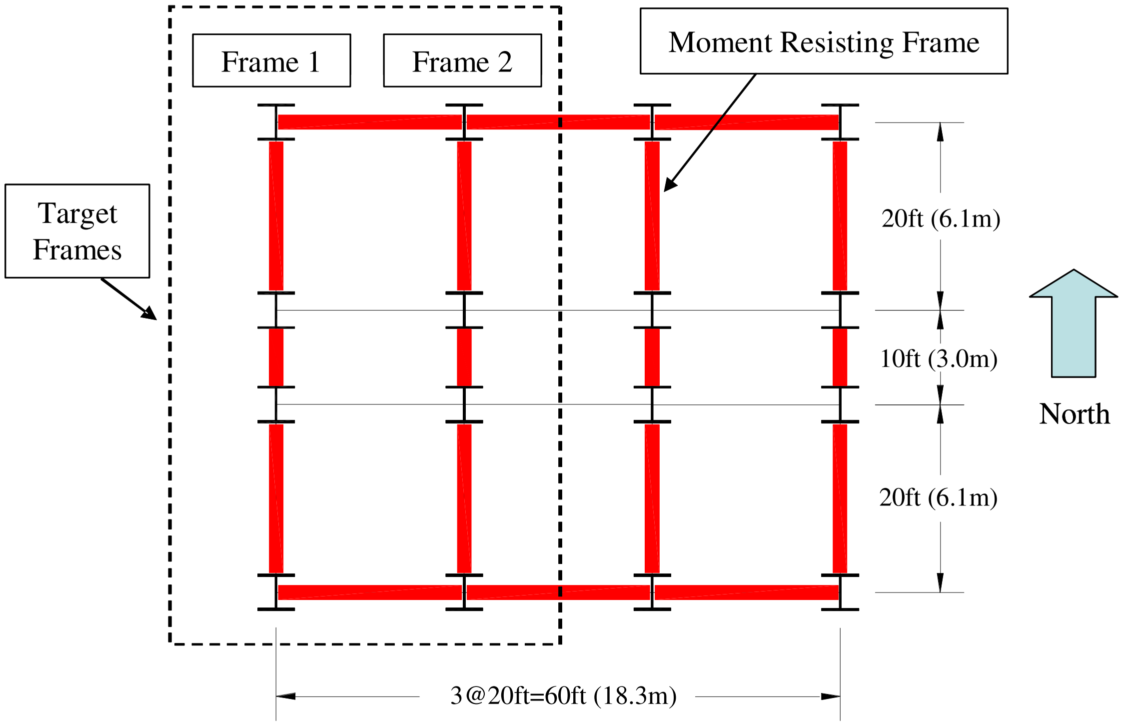


Figure 2 - Building Plan

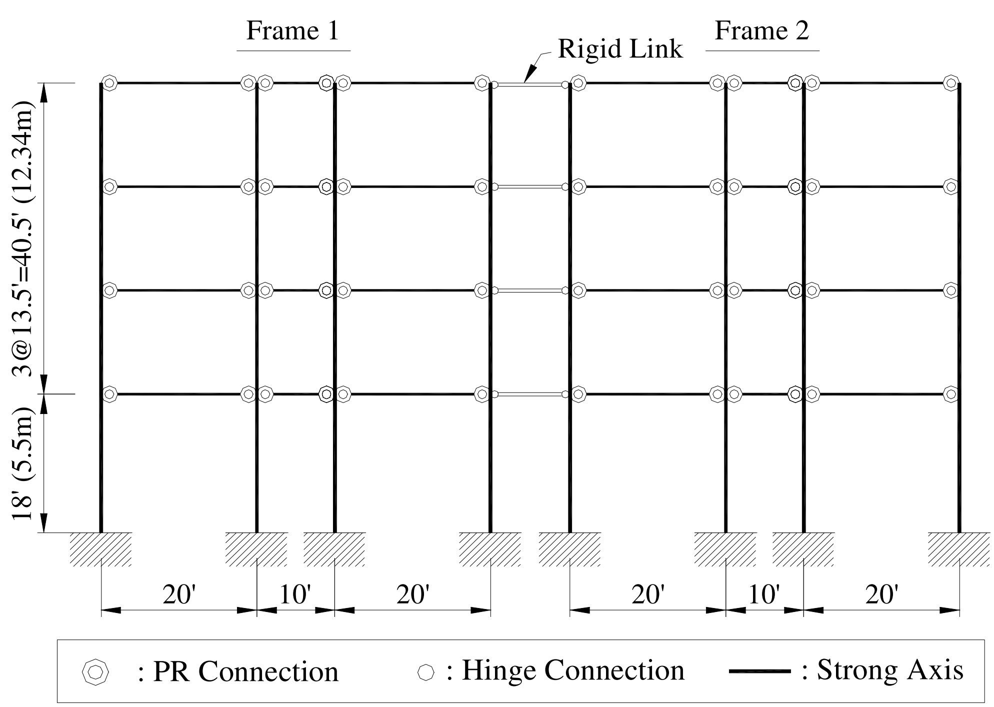


Figure 3 – Analytical model

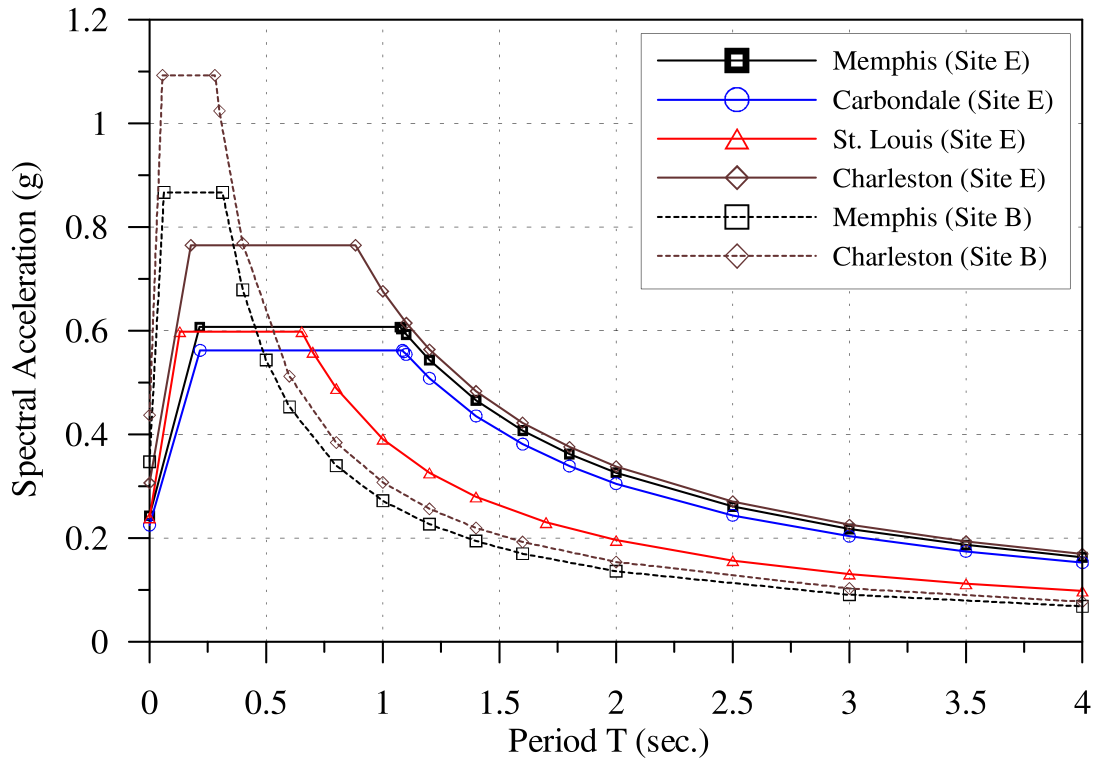


Figure 4 – Design spectra used for frame design

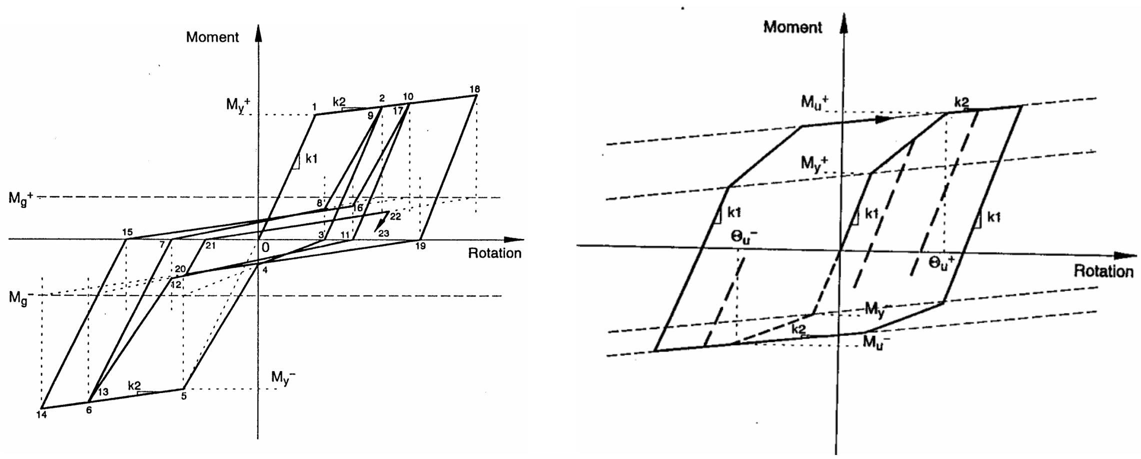
CONNECTION MODELING

Three different types of PR connections – clip angle (CA), T-stub (T), and bolted-flange-plate (BFP) connections – were modeled and utilized, as appropriate, in the moment frames used in these investigations. Clip angle and T-stub connections were used in the Older frames.

Two approaches were used to model the connection behavior, as shown in Table 2. The backbone curve was derived by either curve-fitting to existing tests (labeled as EXP) with similar member sizes or by using the FEMA 273 [9] backbone curves (labeled as FEMA). The hysteretic behavior was modeled using available elements developed by Shi and Foutch [10] for DRAIN2-DX. Figure 5 illustrates the typical behavior modeled, with Model 1 indicating slip at service loads typical of T-stub connections and Model 3 showing fuller hysteretic loops as expected from a BFP connection.

Table 2 - Modeling of PR Connections

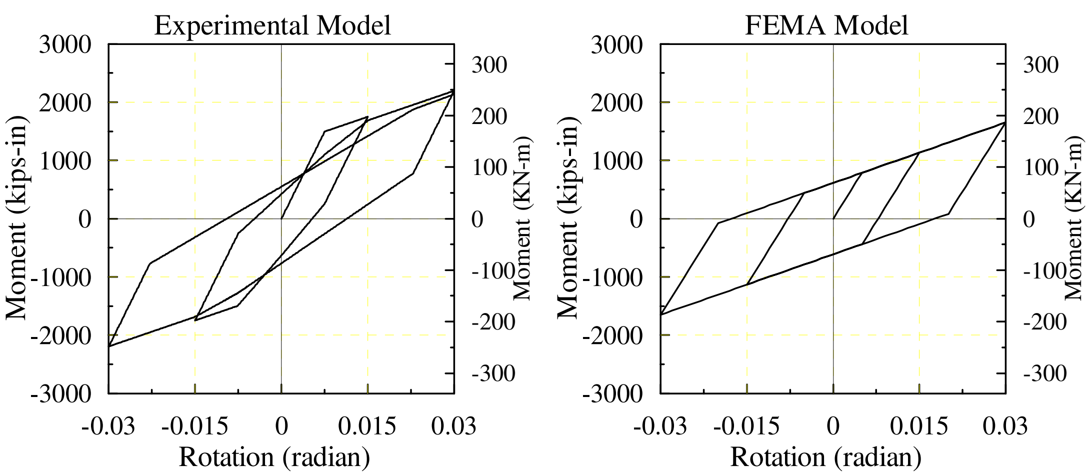
Frame	Applied PR Connection	Modeling method	Hysteresis Model
Older	T-stub	Curve fitting	Model 1
	Clip angle	Curve fitting	Model 1
		FEMA273	Model 2
Intermediate & Modern	T-stub	Curve fitting	Model 1
	Bolted-Flange-Plate	Curve fitting	Model 3



(a) Model 1- Pinching Hysteresis Model (b) Model 3 -Tri-Linear Hysteresis Model

Figure 5 - Connection element models in DRAIN-2DX [9]

The two different modeling methods lead to substantially different results. Figure 6 shows a comparison of the behavior at large displacements for a heavy clip angle connection. The differences in strength and energy dissipation are clear, indicating the generally very conservative nature of the FEMA 273 provisions.



(a) Curve fitting Model (b) FEMA Model

Figure 6 - Comparison of the third cycle for both models at 3% Rotation

The analyses were run following the FEMA 355C [11] studies, which made the following assumptions:

- A mass corresponding to 1.0DL+0.25LL was applied
- The plastic zones in beams and columns were modeled as zero-length hinges.
- The hysteretic behavior at the plastic hinges was described by a bilinear moment-rotation diagram.
- All elements were assumed as having 3% strain hardening.
- The expected rather than the nominal yield strength values were used
- 2% viscous damping was used in first mode and at T=0.2 Section
- Soil-structure interaction was not considered.

RESULTS

Typical results for both pushover and non-linear time history analyses will be illustrated with data from the Older frames. Complete data for the other frames is available in Kim [12].

Pushover Analyses

The expected performance of a structural system can be evaluated by estimating the strength and deformation demands under the design earthquakes by means of static inelastic (pushover) analysis, and comparing these demands to the available capacities at the performance levels of interest. The important performance parameters selected for comparison are the global interstory drift angle (G-ISDA), the drift ratio at each story level, and the inelastic deformations. Figure 7 shows the comparisons of the nonlinear pushover analyses results for the Older frames with two clip angle connection models (O-EXP-CA and O-FEMA-CA) and one T-stub connection model (O-EXP-T). The G-ISDA on the x-axis is defined as the roof displacement divided by total building height.

Figure 7 shows that the strength of the taller Older frames deteriorated more rapidly than that of the shorter frames due to the large P-Δ effects. In addition, the maximum strengths of the 4-, 6-, and 8-story Older frames with T-stub connection model were 4%, 12%, and 15% larger than those of the clip angle connection model, respectively. This result was expected because the T-stub connection model is stiffer and stronger than the clip angle connection model, although the difference had been expected to be larger. For the case of the clip angle connection of the FEMA model, the maximum strengths of all three heights were approximately 20% less than those of the curve fitting or EXP model. The initial stiffness for three frames was almost identical. The 6- and 8-story Older frames with the FEMA model collapsed due to the local connection failure at less than 4% of G-ISDA, while the 4-story frame reached almost 8% of G-ISDA.

Figure 8 shows the profiles of lateral displacements and drift ratios for the Older clip angle connections with a curve-fitting model (EXP-CA). All frames were stable up to 2% G-ISDA level (yielding level), with the maximum story drift ratio at this point typically less than 0.04 rad. After reaching the maximum strength capacity (2% of G-ISDA level), plastic deformations increased significantly and concentrated on the lower story levels. For the 4-story case, large plastic deformations were observed at the 1st story level. The maximum drift ratio for both the 6- and 8-story frames occurred at the 3rd story level at 2% G-ISDA , but it gradually moved down to the 1st story level as plastic deformations increased (4% to 8% of G-ISDA level).

Non-Linear Time History Analyses

Nonlinear time-history (NTH) analyses were performed on the Older frame for four cities in Mid-America utilizing a suite of 30 ground motions with a 2%/50yrs probability of exceedance. The primary response indices were plotted as a median values for each set of ground motions along each story level. Typical results are shown in Figure 9, with the results from Memphis, Carbondale, and St. Louis shown from top to bottom, respectively. The median (□), 84th percentile (◊), and 95th percentile (Δ) values of the maximum ISDA observed from the analyses are plotted on the left part of the figure. The plots on the right represent the median demand of the PR connection rotation (PRCR, ◊), the panel zone deformations (PZD, Δ), and median interstory drift (ISDA, □).

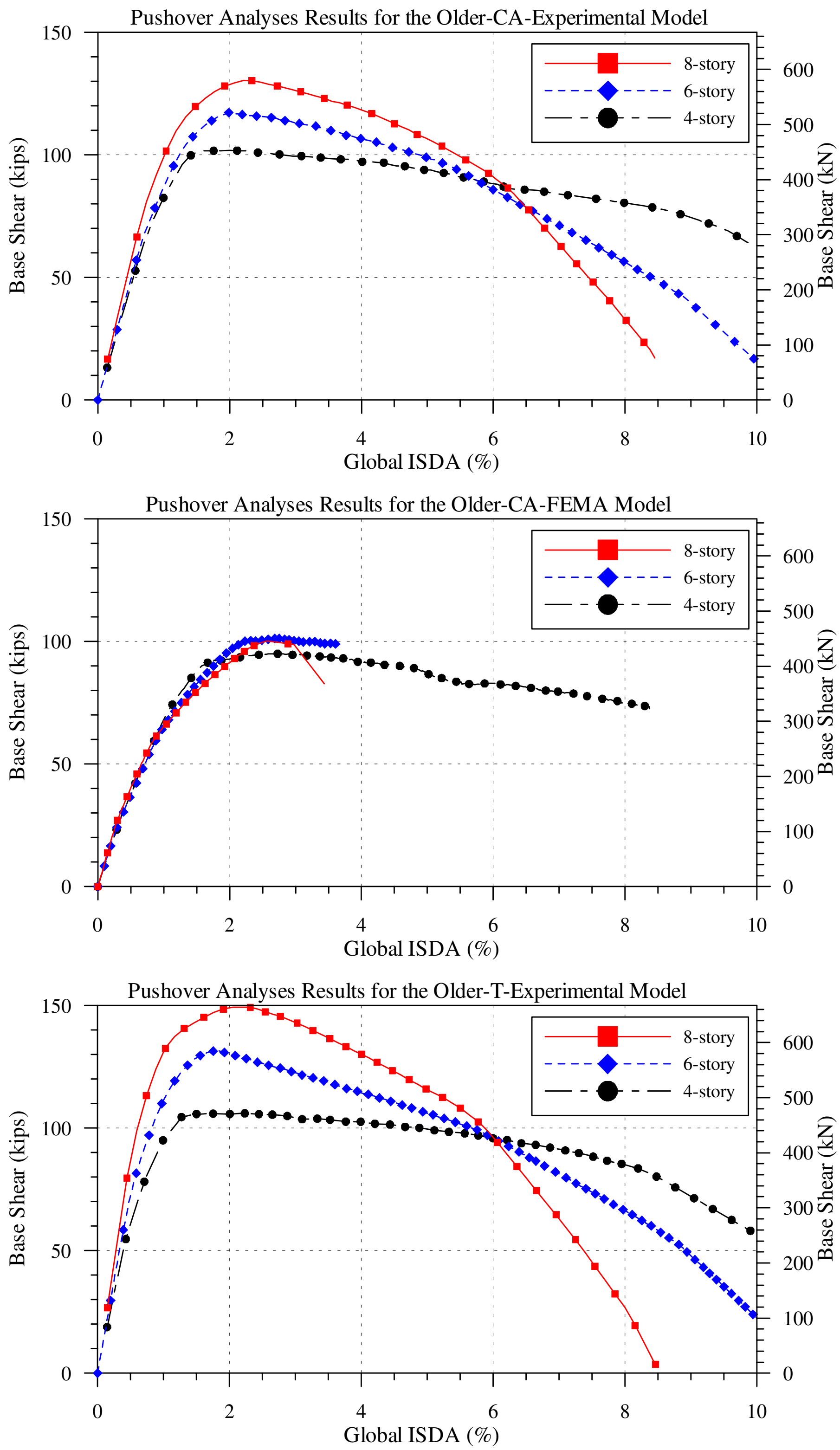


Figure 7 – Comparison of pushover results for the Older Frames

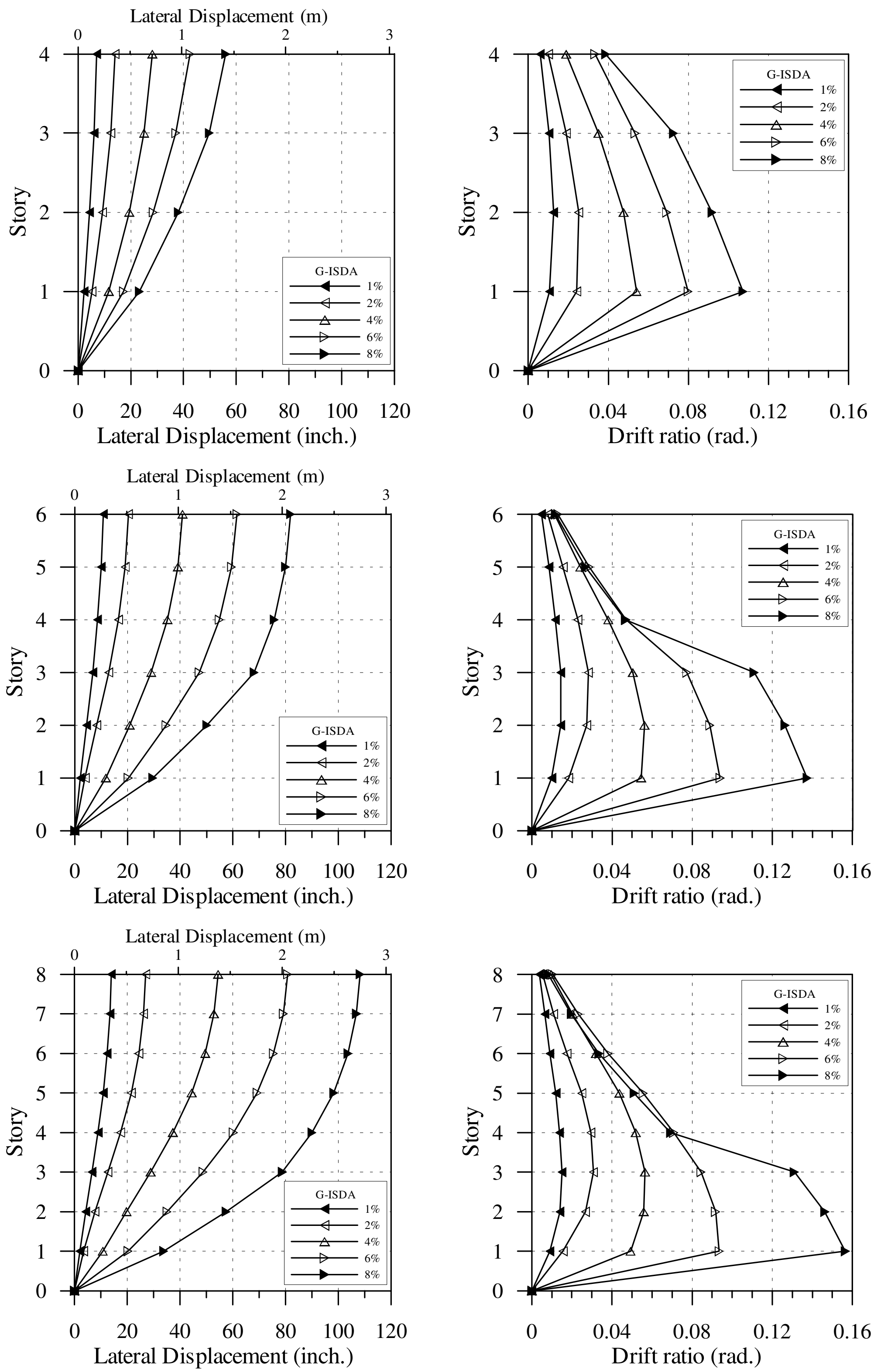


Figure 8 - Profile of the lateral displacements and drift ratios for the Older frames

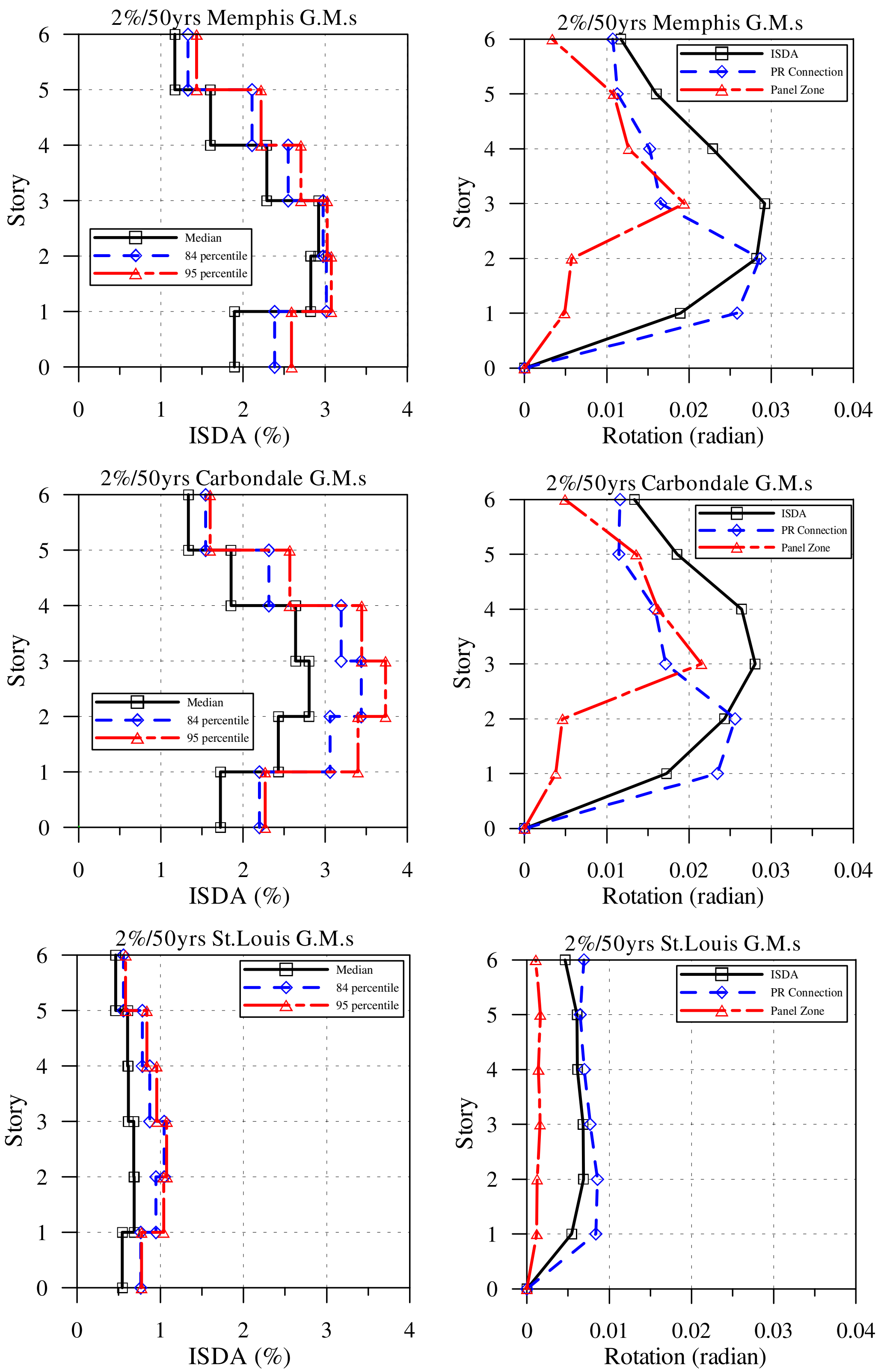


Figure 9 – Summary of non-linear time history analyses for the 6-story Older frame

CONCLUSIONS

Analysis of the large set of data generated in this project for older steel frames in Mid-America indicates that:

(1) The Memphis and Carbondale set of ground motions resulted in larger nonlinear responses than those for the St. Louis ground motions. The scaled Charleston set of ground motions resulted in similar nonlinear response to those for Carbondale and Memphis.
(2) A structure having a short period, such as 4- and 6-story Older frames, is more sensitive to the site characteristics for Mid-America than one having a long period, such as an 8-story frame.
(3) Pushover analyses indicated that the Older frames were stable until a 0.04 rad drift ratio. However, after this drift level was reached, large plastic deformations concentrated on the first or lower story leading to a rapid collapse.
(4) The median PR connection rotations for the Older frame with both clip angle connection models were in the range of 0.015 to 0.030 rad for Memphis and Carbondale, but less than 0.010 for St. Louis.
(5) The differences of the median PR connection rotations between the EXP and FEMA clip angle connection models were noticeable in the 4-story Older frame, but these differences were insignificant in the 6- and 8-story Older frames.
(6) The differences of the median panel zone deformations between the EXP and FEMA clip angle connection models were noticeable. For example, for the 6-story Older frame in Carbondale, the median panel zone deformation of the FEMA clip angle connection model was 66% lower than that of the EXP clip angle connection model.
(7) The median panel zone deformations for the Older frame with T-stub connections were in the range of 0.017 to 0.023 rad for Memphis and Carbondale.
(8) In contrast to the PR connection rotations, the median panel zone deformations for the case of the O-EXP-T were approximately equal to or up to 8% larger than those for the EXP clip angle connection model.

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