Ductility, interstory and displacement of steel buildings with perimeter and spatial moment frames

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SUMMARY:
The linear and nonlinear responses of steel buildings with perimeter moment resisting frames (PMRFs) are estimated and compared with those of buildings with equivalent spatial moment resisting frames (SMRFs). The interstory shears of the buildings with PMRSFs may be significatively larger than those of the buildings with spatial MRSFs. The interstory displacements are larger for the system with SMRFs. The differences, however, are much smaller for the case of displacements. The global and story ductility demands are larger for the steel buildings with PMRFs, implying that the detailing of the connections of this structural system will have to be more stringent than for the building with SMRFs. It can be concluded that the seismic performance of the steel buildings with SMRFs may be superior to that of steel buildings with PMRFs. However, proper detailing of connections has to be taken into account in order the get the required rotations.

Keywords: steel buildings, nonlinear response, perimeter and spatial moment frames, story and global ductility.

1. INTRODUCTION AND OBJECTIVES

Among the different structural systems used to support lateral seismic loading, moment resisting steel frames (MRSFs) are broadly used for the case of steel buildings. They have been popular because they provide maximum flexibility for space utilization and because of their high ductility capacity. The basic structural arrangement of this structural system, however, has significantly changed over the years. From the mid 60s to the mid 70s, at least in USA, most of the connections in steel buildings were fully restrained connections (FRCs), resulting in highly redundant buildings. For the case of weak axis connection, it was the standard practice for many years (FEMA 355C) to frame the beams to the columns by welding the beam flange to a continuity plate which in turn was welded to the web and the flanges of the column. Tests have shown (Rentschler 1980) that this type of weak-axis connection is susceptible to fracture at the weld connecting the beam flange to the continuity plate. However, this problem can be mitigated by using several measures, including extending the continuity plate beyond the column flanges. Because the mitigation procedure is expensive, the standard practice during the recent past (after the 80s) has been to eliminate weak-axis moment connections. Most of the steel buildings with MRSFs built in USA have FRCs only on two frame lines in each direction, usually at the perimeter, and often these frames do not extend over the full plan of the buildings. These frames are used to support the total seismic lateral load while Gravity Frames (GFs), used at the interior, are used to support the gravity loads.

The main advantages of using steel buildings with PMRFs are that they are considered to behave in two dimensions within a three-dimensional structure providing a simpler frame to analyze and design and that using fewer FRCs introduces overall economy in the design since these connections are expensive. However, there are several disadvantages too, some of them are: (1) since the size the girders of the PMRFs is very large, the amount of strain demand placed on the welded connection elements is also too large, making the connections more susceptible to brittle behavior (FEMA 355C); (2) the PMRFs, modeled as plane frames, are usually designed to resist the total lateral seismic loading, ignoring the contribution of the GFs, and (3) because much fewer FRCs are used in comparison with SMRFs, the redundancy of the building is tremendously reduced. The relatively low redundancy of steel buildings with perimeter PMRFs has been appointed by the engineering community as one of the possible causes of their poor seismic performance (Mele et al 2004). In section 2.5.4 of FEMA 350 it is stated "there are several reasons why structures with some redundancy in their structural systems should perform better than structures without such redundancy. Redundant structures, on the other hand, would still retain some significant amount of lateral resistance following failure of a few elements.

In Mexico, it is common to use steel buildings with MRSFs at the perimeter and the interior (RCDF 2004) in both horizontal directions. Due to the large number of FRCs of this system, its redundancy and ductility capacity are expected to be greater than those of the systems with only perimeter PMRFs, although the structural analysis and design is more complicated. Comparison of performance of these two structural systems under the action of severe seismic loads is undoubtedly of great interest to the profession and therefore it is addressed in this research.

Many investigations have been developed regarding the seismic behavior of buildings with MRSFs (Gupta and Krawinkler 2000, Lee and Foutch 2001, Foutch and Yun 2002, Mele et al 2004, LeeRand Foutch 2006, Krishnan et al 2006, Liao et al 2007, Kazantzy et al 2008, Chang et al 2009). The seismic nonlinear response of steel plane frames with MRSFs considering the dissipation of energy has been also studied (Firat and Liu 2004, Reyes-Salazar and Haldar 2000, Reyes-Salazar and Haldar 2001a, Reyes-Salazar and Haldar 2001b, Firat and Liu 2004, Merhabian et al 2005). In spite of the amount of research developed in the and the important contributions of the earlier-mentioned and other studies, the comparison of the performance of three-dimensional steel buildings with PMRFs and SMRFs in terms of redundancy and ductility have not been studied. The seismic responses of these two structural systems are expected to be different since they dynamic characteristics are different too. The estimation and comparison of the linear and nonlinear seismic responses of these structural systems constitute the main objective of the present investigation.

To meet the objectives of the study, the behavior of some steel buildings models are represented as realistically as possible, preferably in 3-D and then estimating responses by exciting them with measured seismic time histories. Specifically, the linear and nonlinear seismic responses, in terms of interstory shears, interstory displacements and ductility, are estimated for steel buildings with PMRFs and compared with those of their equivalent steel buildings with SMRFs.

2. DUCTILITY DEFINITIONS

The ductility parameter plays an important role in the determination of the design seismic forces. It represents the capacity of a structure to dissipate energy. It is particularly important for steel structures since the beneficial effect of ductility is supposed to come from different sources. Although the concept of ductility is constantly used in the profession, at present there is no engineering definition of it in the specifications and codes and there is no unanimity in the profession on how to define it. It is used in an indirect way in design. In a research report (SAC, 1995) it was stated, "Ductility is shown in parentheses to emphasize that there is no definition of ductility in our Specification and Codes but it is always being used. The metallurgical definition of ductility is the ability of a metal to be stressed beyond its yield strength and into its plastic (inelastic) range, with large elongations before rupturing in a ductile mode. An engineering definition of ductility may be needed, as related to moment-resisting frames design and construction."

As stated earlier, the seismic responses of the two structural systems under consideration are also studied in terms of ductility demands. A definition of ductility for multi-degree of freedom (MDOF) systems is adopted here for that purpose. In the context of seismic analysis of single degree of freedom (SDOF) systems, ductility can be conceptually defined as the ratio of the maximum inelastic displacement (D_{max}) to the yield displacement (D_y). D_y can be defined as the displacement of the system when it yields for the first time and D_{max} as the maximum displacement that the system undergoes during the application of the complete earthquake loading. For MDOF systems, however, there is no unanimity in the profession on how to define it. Definitions of story and global ductility proposed by Reyes-Salazar (2002) and used in other investigations (Annan et al. 2009) are used in this study. In these definitions, story ductility (μ) is defined, for each story, as D_{max}/D_y. D_{max} is the maximum interstory lateral displacement after the application of the complete time history of an earthquake and D_y is defined, for all stories, as the maximum interstory lateral displacement when the first plastic hinge is developed in the structure. Global ductility (μ_G) is estimated as the mean value of the story ductilities.

3. MATHEMATICAL FORMULATION

An assumed stress-based finite element algorithm, developed by the authors and their associates (Gao and Haldar 1995, Reyes-Salazar 1997), is used to estimate the nonlinear seismic responses of several steel building models. The procedure estimates the responses in time domain, as accurately as possible by considering material and geometry nonlinearities. In this approach, an explicit form of the tangent stiffness matrix is derived without any numerical integration. Fewer elements can be used in describing a large deformation configuration without sacrificing any accuracy, and the material nonlinearity can be incorporated without losing its basic simplicity. It gives very accurate results and is very efficient compared to the commonly used displacement-based approaches. The procedure and the algorithm have been extensively verified using available theoretical and experimental results (Reyes-Salazar and Haldar 2001a, Reyes-Salazar and Haldar 2001b).

4. STRUCTURAL MODELS

4.1 Buildings with perimeter moment resisting steel frames (PMRFs)

Several steel model buildings were designed, as part of the SAC steel project (FEMA, 355c). They were 3-, 9- and 20- story buildings which were designed according to the code requirements for the following three cities: Los Angeles (UBC, 1994), Seattle (UBC, 1994) and Boston (BOCA, 1993). The 3- and 9-story buildings, representing Los Angeles area and the Pre-Northridge Designs, are considered in this study to address all the issues raised earlier. They will be denoted hereafter as Models SAC1 and SAC2, respectively. The beam and columns sections are given in Table 1. Additional information for the models can be obtained from the SAC steel project reports (FEMA, 355C). In this study, the frames are modeled as MDOF systems. Each column is represented by one element and each girder of the PMRFs is represented by two elements, having a node at the mid-span. Each node is considered to have six degrees of freedom. The total number of degrees of freedom is 846 and 3408, for Models SAC1 (3-level) and SAC2 (10-level), respectively. The models are excited by twenty recorded earthquake motion in time domain, recorded at the following stations: Paraíso, Mammoth H.S., GymConvict Creek, Infiernillo N-120, La Unión, Relaciones Ext. 1, Relaciones Ext. 2, Long Valley Dam, K2-02, Redwood City, MT:Kalispell, Villita, Hall Valley 1, Hall Valley 2, K2-04, Dauville F.S. CA, Pleasant Hill F.S. 1, Pleasan Hill F.S. 2, Valdez City Hall and Hollister City Hall.

4.2 Buildings with equivalent spatial moment resisting steel frames (SMRFs)

The equivalent models with SMRFs are designed in such a way that their fundamental period, total mass and lateral stiffness are fairly the same as those of the corresponding buildings with PMRFs. The member properties of the equivalent buildings are selected for one direction, say the N-S directions, and then, in order to keep *the equivalence* of the two systems, the same properties are assigned to the other direction. Two cases are considered for the equivalent models. In the first case, the sections of beams and columns of the SMRFs frames are selected by considering the beams and columns of the PMRFs oriented in the direction under consideration. In the second case, the beams and columns of the SMRFs are generated by additionally considering the perpendicular PMRFs. It must be noted that the columns of these frames bend with respect to their minor axis. The ratio of moments of inertia, or plastic moments, between beams and columns was tried to keep as close as possible for the two structural systems. The same was considered for the case of interior and exterior columns. The equivalent models for the first case are referred, in general, as EQ1 Models and, in particular, as Models EQ11 and EQ12 for the 3- and 10-level buildings, respectively, while for the second case they are referred, in general, as EQ2 models and in particular as Models EQ21 and EQ22 for the 3- and 10-level buildings. The resulting sections are shown in Table 2.

5. RESULTS IN TERMS OF INTERSTORY SHEARS

5.1. SAC and EQ1 models

The average interstory shears are estimated for the SAC models and compared to those of their corresponding equivalent EQ1 models. The shear ratio V_1, defined as V_{SAC1}/V_{EQ1}, is introduced to make the comparison. For a given direction and story, V_{SAC1} will represent the average interstory shear resisted by all the frames of the SAC models for the story under consideration and V_{EQ1} will represent the same , but for the EQ1 models. The buildings remain essentially elastic when subjected to any of the earthquakes. All the earthquake time histories are normalized with respect to their maximum peak ground acceleration. The recorded seismic components are applied along the principal structural axes; the horizontal component with the major peak acceleration is applied in the *N-S* direction and the other in the *E-W* direction. The vertical component is also considered.

Table 1. Beam and columns sections for SAC Models

MODEL	MOMENT RESISTING FRAMES				GRAVITY FRAMES		
	STORY	COLUMNS		GIRDERS	COLUMNS		BEAMS
		EXTERIOR	INTERIOR		BELOW	OTHERS	
3-LEVEL (SAC11)	1\2	W14x257	W14x311	W33X118	W14x82	W14x68	W18x35
	2\3	W14x257	W14x312	W30X116	W14x82	W14x68	W18x35
	3\Roof	W14x257	W14x313	W24X68	W14x82	W14x68	W16x26
10-LEVEL (SAC12)	-1/1	W14x370	W14x500	W36x160	W14x211	W14x193	W18x44
	1/2	W14x370	W14x500	W36x160	W14x211	W14x193	W18x35
	2/3	W14x370	W14x500,W14x455	W36x160	W14x211,W14x159	W14x193,W14x145	W18x35
	3/4	W14x370	W14x455	W36x135	W14x159	W14x145	W18x35
	4/5	W14x370,W14x283	W14x455,W14x370	W36x135	W14x159,W14x120	W14x145,W14x109	W18x35
	5/6	W14x283	W14x370	W36x135	W14x120	W14x109	W18x35
	6/7	W14x283,W14x257	W14x370,W14x283	W36x135	W14x120,W14x90	W14x109,W14x82	W18x35
	7/8	W14x257	W14x283	W30x99	W14x90	W14x82	W18x35
	8/9	W14x257,W14x233	W14x283,W14x257	W27x84	W14x90,W14x61	W14x82,W14x48	W18x35
	9/Roof	W14x233	W14x257	W24x68	W14x61	W14x48	W16x26

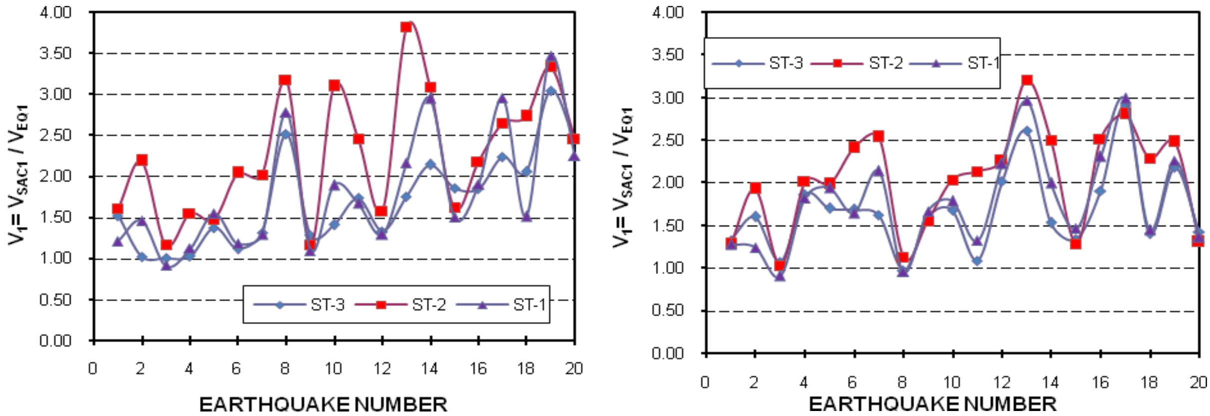
Typical values of the V_1 parameter are shown in Figs. 1a and 1b for the 3- level model and the *N-S* and *E-W* directions. The symbol *ST* is used in the figures to represent the word “story”. It is observed that the V_1 values significantly vary from one earthquake to another, from on model to another and from one story to another without showing any trend ranging from 0.8 to 3.8. The results are similar for the 10-level model. In general the V_1 values are larger for the *N-S* that for the *E-W* direction. The most important observation that can be made is that the V_1 parameter is larger than unity in practically all the cases indicating that the interstory shears are larger for the buildings with PMRFs.

To study the effect of inelastic behavior on the V_1 parameter, the actual time histories were scaled up so that yielding was produced in all the models. All the actual time histories were scaled up to develop a maximum average interstory drift of about 2% by the trial and error procedure. It was observed that about ten to twenty five plastic hinges were formed in the models.

Table 2. Beam and columns sections for the equivalent Models

MODEL	EQ1 MODELS				EQ2 MODELS		
	STORY	COLUMNS		GIRDERS	COLUMNS		GIRDERS
		EXTERIOR	INTERIOR		EXTERIOR	INTERIOR	
3-LEVEL	1\2	W14 X 74	W14 X 90	W24 X 55	W16 X 67	W14 X 109	W12 X 170
	2\3	W14 X 74	W14 X 90	W21 X 57	W16 X 67	W14 X 109	W14 X 120
	3\Roof	W14 X 74	W14 X 90	W14 X 43	W16 X 67	W14 X 109	W16 X 40
10-LEVEL	-1/1	W14 X 159	W14 X 211	W27 X 94	W18 X 143	W21 X 166	W24 X 162
	1/2	W14 X 159	W14 X 211	W27 X 94	W18 X 143	W21 X 166	W24 X 162
	2/3	W14 X 159	W14 X 211	W27 X 94	W18 X 143	W21 X 166	W24 X 162
	3/4	W14 X 159	W14 X 193	W24 X 94	W18 X 143	W21 X 147	W21 X 166
	4/5	W14 X 159	W14 X 193	W24 X 94	W18 X 143	W21 X 147	W21 X 166
	5/6	W14 X 109	W14 X 159	W24 X 94	W21 X 93	W27 X 84	W21 X 166
	6/7	W14 X 109	W14 X 159	W24 X 94	W21 X 93	W27 X 84	W21 X 166
	7/8	W14 X 99	W14 X 109	W24 X 55	W14 X 145	W18 X 106	W24 X 68
	8/9	W14 X 99	W14 X 109	W21 X 50	W14 X 145	W18 X 106	W12 X 152
	9/Roof	W14 X 90	W14 X 99	W16 X 45	W24 X 62	W18 X 97	W16 X 67

Plots similar to those previously discussed are then developed, but are not shown. However, as for the elastic behavior case, it is observed that the V_1 values significantly vary from one earthquake to another, from on model to another and from one story to another without showing any trend, that the V_1 values are larger for the *N-S* than for the *E-W* direction, and that they are larger than unity in practically all of the cases. The values of V_1 are averaged over all the earthquakes, the statistics are summarized in Table 3. As observed from individual plots, the statistics indicate the interstory shears may be significantly larger for the buildings with PMRFs. The mean values are similar for both models and levels of deformation, but they are larger for the *N-S* than for the *E-W* direction. The uncertainty in the estimation of V_1 in terms of the coefficient of variation (COV) is significant in many cases, being quite similar for both models, levels of deformation and structural directions.



e) *N-S* direction f) *E-W* direction

Fig. 1. Values of the V_1 parameter, 3-level building elastic behavior

5.2. SAC and EQ2 models

The comparison between the interstory shears of the *SAC* and the *EQ2* models is made in terms of the V_2 parameter which is defined as V_{SAC1} / V_{EQ2}. Similar plots and statistics to those of V_1 are also developed for V_2, but are not presented. Many of the observations made for the V_1 ratio also apply to the V_2. The only additional observations that can be mentioned are that the mean values of V_2 and the uncertainty in its estimation may be significantly larger for the 10- than for the 3-level building and that these parameters are quite similar for both structural directions. It is also observed that the mean and COV of V_2 are in general smaller than those of V_1. It indicates that considering the perpendicular MRSFs in the generation of the equivalent models has an important effect on the structural response.

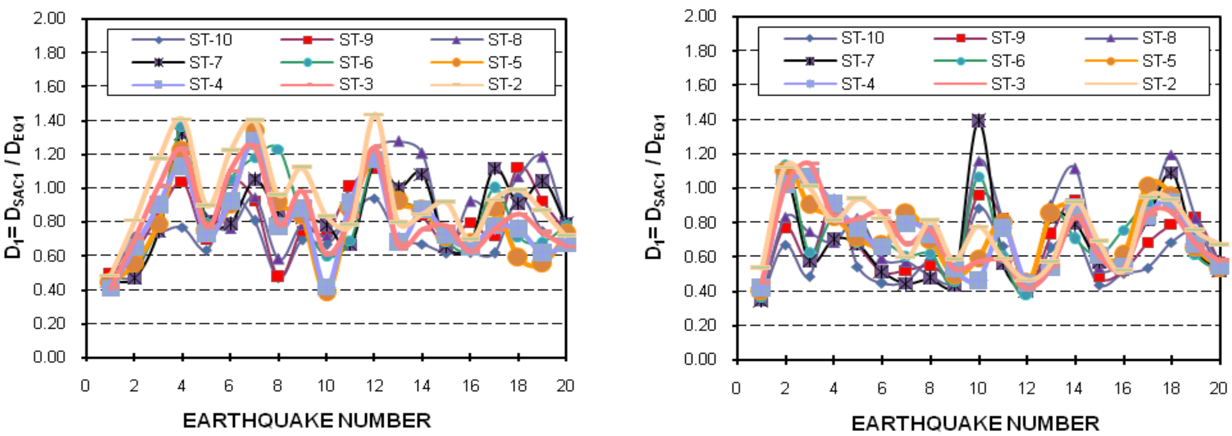
Table 3. Statistics for the V_1 and D_1 ratios

MODEL		STORY	STATISTICS OF V_1						STATISTICS OF D_1					
			N-S direction			*E-W* direction			*N-S* direction			*E-W* direction		
			Mean	SD	CV	Mean	SD	CV	Mean	SD	CV	Mean	SD	CV
3-LEVEL	ELASTIC	3	1.71	0.57	0.33	1.68	0.48	0.29	0.97	0.28	0.29	0.86	0.27	0.31
		2	2.27	0.77	0.34	2.04	0.60	0.30	0.81	0.27	0.34	0.69	0.22	0.32
		1	1.81	0.73	0.40	1.79	0.58	0.32	0.89	0.32	0.36	0.81	0.27	0.33
	INELASTIC	3	1.68	0.57	0.34	1.62	0.48	0.30	0.96	0.26	0.27	0.87	0.27	0.31
		2	2.21	0.75	0.34	1.98	0.59	0.30	0.81	0.27	0.33	0.70	0.21	0.31
		1	1.76	0.71	0.40	1.75	0.57	0.33	0.89	0.34	0.38	0.80	0.26	0.33
10-LEVEL	ELASTIC	10	1.40	0.20	0.14	1.24	0.29	0.24	0.71	0.13	0.19	0.59	0.16	0.27
		9	1.85	0.39	0.21	1.54	0.38	0.24	0.83	0.19	0.23	0.66	0.18	0.27
		8	1.95	0.51	0.26	1.67	0.49	0.29	0.88	0.24	0.28	0.73	0.24	0.33
		7	1.87	0.45	0.24	1.68	0.58	0.34	0.86	0.23	0.27	0.67	0.26	0.39
		6	1.92	0.51	0.26	1.76	0.46	0.26	0.85	0.25	0.29	0.70	0.21	0.30
		5	1.82	0.52	0.29	1.73	0.38	0.22	0.81	0.25	0.31	0.73	0.19	0.26
		4	1.83	0.47	0.26	1.73	0.42	0.24	0.80	0.22	0.28	0.71	0.19	0.27
		3	1.92	0.50	0.26	1.75	0.43	0.24	0.84	0.23	0.27	0.71	0.19	0.27
		2	1.85	0.48	0.26	1.59	0.36	0.23	0.96	0.26	0.26	0.75	0.19	0.25
	INELASTIC	10	1.79	0.37	0.21	1.57	0.42	0.27	0.70	0.13	0.19	0.60	0.18	0.29
		9	1.87	0.46	0.25	1.68	0.47	0.28	0.80	0.20	0.25	0.69	0.21	0.30
		8	1.80	0.34	0.19	1.68	0.53	0.32	0.84	0.28	0.33	0.76	0.27	0.36
		7	1.87	0.42	0.22	1.78	0.47	0.26	0.83	0.25	0.30	0.68	0.29	0.43
		6	1.80	0.50	0.28	1.77	0.41	0.23	0.84	0.26	0.31	0.71	0.25	0.36
		5	1.84	0.45	0.25	1.75	0.41	0.23	0.80	0.24	0.30	0.74	0.20	0.28
		4	1.90	0.48	0.25	1.76	0.40	0.23	0.80	0.21	0.26	0.70	0.20	0.29
		3	1.81	0.46	0.26	1.61	0.34	0.21	0.85	0.23	0.27	0.71	0.20	0.29
		2	1.79	0.37	0.21	1.57	0.42	0.27	0.97	0.28	0.29	0.76	0.20	0.26

6. RESULTS IN TERMS OF INTERSTORY DISPLACEMENTS

6.1. SAC and EQ1 models

The D_1 parameter is used to compare the interstory displacements of the *SAC* models with those of the EQ1 models. D_1 is defined as D_{SAC1} / D_{EQ1} where the terms of this ratio have a similar meaning than those of the case of shear, except that they now represent interstory displacements. The values of D_1 are given in Figs 2a and 2b for the 10-level building and elastic behavior. The results resemble those of interstory shears in the sense that the values significantly vary without showing any trend, but they are different in another sense since the values of these parameters are smaller than unity in most of the cases indicating that the interstory displacements are larger for the buildings with spatial MRSF. The differences, however, are much larger for the case of shears than for displacements.



c) *N-S* direction

d) E-*W* direction

Fig. 2. Values of the D_1 parameter, 10-level building elastic behavior

Plots for D_1 and inelastic behavior are also developed but are not shown. The statistics for both, elastic and inelastic behavior are presented in Table 3. The statistics confirm what observed for individual plots: the interstory displacements are larger for the buildings with SMRFs. It is observed that the mean values and the uncertainty in the estimation are larger for the 3- than for the 10-level model, but they are quite similar for elastic and inelastic behavior.

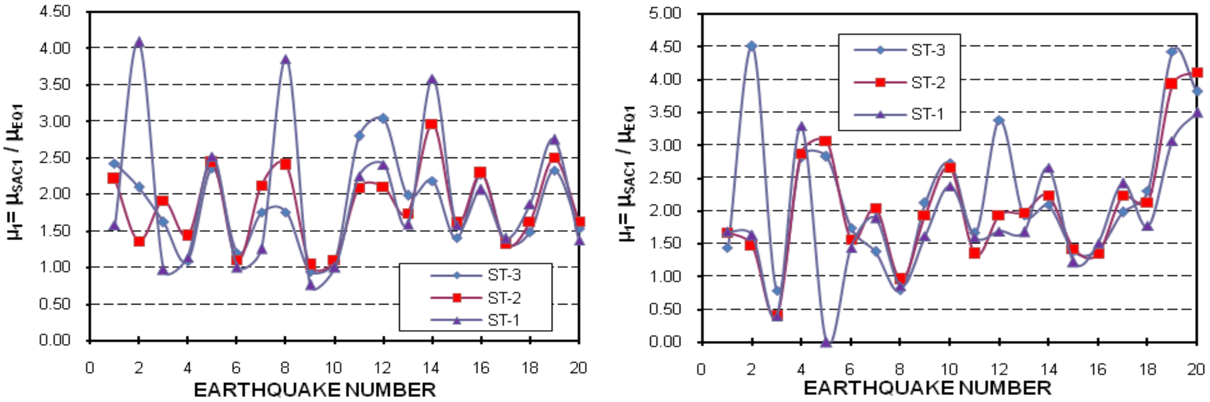
6.2. SAC and EQ2 models

The comparison between the interstory displacements of the *SAC1* and the *EQ2* models is made in terms of the D_2 parameter, which is defined as D_{SAC1} / D_{EQ2}. Plots and statistics for D_2 are developed but are not shown. Results indicate, however, that in general, the mean values of D_2 are larger than unity in most of the cases indicating that the values of the interstory displacements are larger for the EQ2 models. The mean values are larger for the 3- than for the 10-level building and quite similar for elastic and inelastic behavior. For the case of the 3-level building the mean values are much larger than unity for the upper story while for the other stories they are close to or smaller than unity. The uncertainty in the estimation is similar for elastic and inelastic behavior, but larger for the 10- than for the 3-level building. Unlike the case of the comparison of the V_1 and V_2 parameters, the mean values of the D_1 and D_2 ratios are quite similar.

7. RESULTS IN TERMS OF DUCTILITY

7.1. Story ductility (μ)

The results in terms of story ductility demands are estimated for all the structural systems under consideration and compared each other. The ratios $\mu_{1=}\ \mu_{SAC/}\ \mu_{EQ1}$ and $\mu_{2=}\ \mu_{SAC/}\ \mu_{EQ2}$ are used for this purpose. The parameters μ_{SAC}, μ_{EQ1} and μ_{EQ2} represent the story ductility demands for the *SAC*, *EQ1* and *EQ2* models, respectively. The results for the μ_1 ratio are presented in Figs 3a and 3b, for the 3-level building. As for the case of the D_1, D_2, V_1 and V_2 parameters, the μ_1 ratio varies from one earthquake to another and from one story to another without showing any trend. Most of the observations made for this model also apply to the 10-level building. However, the μ_1 values are larger for the 3-level building. The most important observation that can be made is that the values of μ_1 are, in general, larger than unity in most of the cases indicating that the ductility demands are larger for the steel buildings with PMRFs; values larger than 3 are reached in many cases. The implication of this is that, since larger ductility demands are imposed for the same level of earthquake loading, the detailing of the connections of this structural system will have to be more stringent than for the building with SMRFs.



a) *N-S* direction b) E-*W* direction

Fig. 3. Values of the μ_1 parameter, 3-level building elastic behavior

The statistics of μ_1 f are presented in Table 4. As commented before for individual plots, it is observed that in general the mean values are, general larger that unity and larger for the 3- than for the 10-level building. For the 3-level building the mean values are larger for the *E-W* direction while for the 10-level buildings they are larger for the *N-S* direction. The uncertainty in the estimation is large with values of COV of up to 0.60 in some cases and it is larger for the *E-W* direction. Similar plots to those of μ_1 are also developed for μ_2, but are not shown, only their statistics are discussed (Table 4). As for the case of the μ_1 ratio, the mean values of μ_2 are larger than unity in most of the cases, indicating again that the ductility demands are larger for the building with perimeter MRSFs. However, unlike the case of μ_1, the mean values of μ_2 are larger for the 10-level building. The uncertainty in the estimation of μ_2 is similar for the 3- and 10-level buildings, but larger for the *E-W* than for the *N-S* direction. The only additional observation that can be made is that, in general the mean values of μ_1 are larger for the case of the 3-level building while for those of μ_2 resulted larger for the 10-level building.

Table 4. Statistics for the μ_1 and μ_2 parameters

MODEL	STORY	Statistics of μ_1						Statistics of μ_2					
		N-S direction			*E-W* direction			*N-S* direction			*E-W* direction		
		Mean	SD	CV	Mean	SD	CV	Mean	SD	CV	Mean	SD	CV
3-LEVEL	3	1.84	0.59	0.32	2.28	1.08	0.48	0.94	0.22	0.23	1.17	0.46	0.39
	2	1.85	0.54	0.29	2.06	0.92	0.44	1.01	0.18	0.18	1.08	0.23	0.22
	1	1.95	0.99	0.51	1.92	0.78	0.41	1.06	0.34	0.37	1.09	0.28	0.26
10-LEVEL	10	0.96	0.37	0.39	0.77	0.38	0.48	1.42	0.46	0.32	1.34	0.51	0.38
	9	1.03	0.44	0.42	0.82	0.46	0.56	1.35	0.36	0.27	1.38	0.51	0.37
	8	1.17	0.47	0.40	0.94	0.58	0.62	1.29	0.32	0.25	1.49	0.55	0.37
	7	1.20	0.52	0.44	0.89	0.55	0.61	1.35	0.39	0.29	1.18	0.34	0.29
	6	1.14	0.45	0.39	1.02	0.46	0.45	1.39	0.36	0.26	1.19	0.31	0.26
	5	1.15	0.50	0.43	1.09	0.59	0.54	1.35	0.35	0.26	1.41	0.44	0.31
	4	1.21	0.47	0.38	1.07	0.54	0.50	1.38	0.45	0.33	1.42	0.48	0.34
	3	1.22	0.50	0.41	1.04	0.45	0.44	1.42	0.39	0.28	1.24	0.39	0.31
	2	1.22	0.49	0.40	1.08	0.51	0.47	1.41	0.39	0.28	1.17	0.40	0.34

Table 7. Values of the μ_{G1} and μ_{G2} parameters

EARTHQUAKE	μ_{G1}				μ_{G2}			
	3-LEVEL		*10-LEVEL*		*3-LEVEL*		*10-LEVEL*	
	N-S	*E-W*	*N-S*	*E-W*	*N-S*	*E-W*	*N-S*	*E-W*
1	2.08	1.59	0.48	0.41	2.11	1.48	2.27	1.41
2	2.26	2.42	0.78	1.19	1.01	1.79	1.33	2.12
3	1.46	0.48	1.37	0.94	0.74	1.37	1.12	1.28
4	1.22	3.00	5.67	6.28	0.65	0.69	1.50	1.45
5	2.44	2.61	0.91	1.04	1.32	1.31	0.98	1.22
6	1.10	1.58	1.37	0.38	0.91	0.99	0.88	0.63
7	1.67	1.76	1.34	1.15	0.82	0.75	1.32	0.77
8	2.53	0.87	1.08	1.00	0.92	0.88	1.07	1.33
9	0.92	1.89	0.95	0.67	1.05	0.71	1.70	3.51
10	1.06	2.59	5.69	1.22	0.87	0.91	1.31	1.55
11	2.34	1.52	1.07	1.02	1.42	1.42	1.72	1.32
12	2.49	2.32	1.85	0.42	1.00	1.09	1.62	1.04
13	1.77	1.86	1.08	0.54	0.89	0.87	1.44	0.94
14	2.93	2.33	1.57	2.41	1.24	1.49	1.50	2.55
15	1.52	1.35	0.66	1.25	1.03	1.02	1.03	1.25
16	2.22	1.43	5.77	1.87	0.96	1.13	1.43	1.35
17	1.35	2.21	0.95	1.15	0.54	1.12	1.36	1.04
18	1.66	2.07	1.32	1.24	0.96	0.90	1.39	1.13
19	2.52	3.81	0.90	0.47	1.04	0.56	1.67	2.53
20	1.50	3.81	1.10	0.27	0.91	0.94	1.26	0.57
Mean	**1.85**	**2.07**	**1.80**	**1.25**	**1.00**	**1.08**	**1.40**	**1.45**
SD	**0.58**	**0.85**	**1.72**	**1.30**	**0.33**	**0.32**	**0.34**	**0.78**
CV	**0.32**	**0.41**	**0.96**	**1.04**	**0.33**	**0.30**	**0.25**	**0.54**

7.2. Global ductility (μ_G)

The values of global ductility, as defined earlier, are calculated and presented in Table 5. In this discussion, μ_{G1} represents the ratios of global ductility of the SAC and EQ1 models while μ_{G2} represent the same ratio for the SAC and EQ2 models. The results confirm what concluded for the ratios of story ductility demands: the global ductility demands are, in general, larger for the structural buildings with PMRFs. This difference in global ductility demands is more significant for the 3-level building for the case of the SAC and EQ1 models (μ_{G1}) while it is more significant for the 10-level building for the case of the SAC and EQ2 models (μ_{G2}). The uncertainty in the estimation of global ductility ratios is larger for μ_{G1} than for μ_{G2}.

8. CONCLUSIONS

The linear and nonlinear seismic responses are estimated for steel buildings with perimeter moment resisting steel frames (PMRFs) and compared with those of their equivalent steel buildings with spatial moment resisting steel frames (SMRFs). The equivalence between the two structural systems is established in terms of total mass, stiffness and lateral strength. Therefore, both systems have approximately the same fundamental period. The comparison is expressed in terms of interstory shears, interstory displacements and ductility. The numerical study indicates that the shear ratio, defined as the quotient of the interstory shears of the buildings with PMRFs and those of the buildings with SMRFs, is larger than unity indicating that the interstory shears are larger for the first structural system. Values of the interstory shear ratio of up to 3 are observed for some cases. This observation is valid for both models and levels of deformation. The shear ratio is slightly different for elastic and inelastic behavior, but they are larger for the *N-S* than for the *E-W* direction. The uncertainty in the estimation of the shear ratio in terms of the coefficient of variation (COV) is significant in many cases, being quite similar for both buildings, levels of deformation and structural directions. Values of up to 0.40 are observed in some cases. It is observed that the results may be quite different when the perpendicular MRSFs are considered in the generation of the equivalent buildings with SMRFs, indicating an important effect of it on the structural response. Unlike the case of interstory shears, the ratio of interstory displacements is smaller than unity in most of the cases indicating that the interstory displacements are larger for the buildings with SMRFs. The differences, however, are much larger for the case of shears than for displacements. The effect of the perpendicular MRSFs is much smaller for the case of displacements than for base shear. The global and story ductility demands are larger for the steel buildings with PMRFs. The implication of this is that, since larger ductility demands are imposed on the building with PMRFs for the same level of earthquake loading, the detailing of the connections will have to be more stringent for this structural system. It can be concluded that the seismic performance of the steel buildings with SMRFs may be superior that that of steel buildings with PMRFs. However, proper detailing of the connections has to be taken into account in order the get the required rotations.

ACKNOWLEDGEMENTS

This paper is based on work supported by La Universidad Autónoma de Sinaloa under grant PROFAPI 2009/146 and by El Consejo Nacional de Ciencia y Tecnología (CONACyT) under grant 50298-J. Any opinions, findings, conclusions, or recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of the sponsors.

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