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Seismic response estimation of steel Buildings with deep columns and PMRF

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ABSTRACT

The responses of steel buildings with perimeter moment resisting frames (PMRF) with W14 columns are estimated and compared with those of buildings with deep columns (W27), which are selected according to two criteria: equivalent resistance and equivalent weight. It is shown that buildings with W27 columns have no problems of lateral torsional, local or shear buckling in panel zone. Whether the response is larger for W14 or W27 columns, depends on the level of deformation, the response parameter and the structural modeling under consideration. Modeling the buildings as two-dimensional structures result in an overestimation of the response. The axial load on columns may be significantly larger for the buildings with W14 columns. The interstory displacements are always larger for W14 columns, particularly for equivalent weight and plane models, implying that using deep columns helps to reduce interstory displacements, which is an important parameter considered in building design codes. This is particularly important for tall buildings where the design is usually controlled by the *drift limit state*. The interstory shears in interior gravity frames (GF) are significantly reduced when deep columns are used. This helps to counteract the no conservative effect that results in design practice, when lateral seismic loads are not considered in GF of steel buildings with PMRF. Thus, the behavior of steel buildings with deep columns, in general, may be superior to that of buildings with W14 columns, using less weight and representing, therefore, a lower cost.

1. INTRODUCTION

The devastating effects caused by large-scale seismic events, occurred in several parts of the world during the last decades, have originated an intensification of earthquake engineering research in recent years. Different structural systems are continuously studied to improve the structural behavior under the action of severe seismic loads. In the case of steel buildings, moment resisting frames (MRF) are widely

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used due to their great ductility capacity. In some developed countries, like United States, the common practice in these buildings is to use two MRF in each direction, usually located at the perimeter (PMRF) and gravity frames (GF) at the interior. The first are designed to resist the total seismic load and the second the gravity loads. For analysis and seismic design purposes, the buildings are modeled as two-dimensional (2D) structures, but in reality what we have is a three-dimensional (3D) structure. Modeling these buildings as planes frames may not represent the real behavior of the structure, since the participation of some elements is not considered and the contribution of some vibration modes is ignored. Besides, the properties in terms of stiffness, mass distribution, natural frequencies and energy dissipation characteristics for the 2D and 3D models of these structures can be very different. Therefore, their seismic responses are expected to be very different too.

In design practice of steel buildings with MRF, the use of deep columns is not common. The reported studies about the behavior of steel buildings with this type of structural system are, mainly, for the case of W14 columns or smaller. However, in many cases, the design requires higher bending and shear stiffness to control the drifts in such a way that the use of larger columns (W24 or higher) could result in more economical designs. Therefore, it is of interest to study the behavior of the structural system under consideration with deep columns. The main objective of this research is to estimate the seismic responses of steel buildings with PMRF with W14 columns and compare them with those of the same buildings but using W27 columns in the PMRF. The comparison is made in terms of individual local and global response parameters as well as in terms of multiple local response parameters. Two- and three-dimensional structural representations are considered.

2. LITERATURE REVIEW

The study of the seismic behavior of steel buildings with MRF has been of particular interest to the civil engineering profession. Gupta and Krawinkler (2000) studied the behavior of several models designed according to the design provisions of some United States cities. Lee and Foutch (2001) studied the seismic behavior of 26 post-Northridge buildings that represent typical steel MRF buildings, subjected to sets of 20 SAC ground motions representing the 2/50 and 50/50 hazard levels. They concluded that all of the post-Northridge buildings exhibit a high confidence of performing. Foutch and Yun (2002) investigated the accuracy of simple nonlinear as well as more detailed modeling methods used in the design of MRF. They showed that the model which incorporates clear length dimensions between beams and columns, panel zones and an equivalent gravity bay without composite action from the slab, could be a practical model with good accuracy. Mele et al. (2004) compared the seismic behavior of steel buildings with perimeter MRF with the seismic behavior of steel buildings with spatial MRF, concluding that the response of the two systems is similar, in terms of local and global response parameters.

In another study, Lee and Foutch (2006) studied the seismic behavior of 3-, 9-, and 20-story MRF designed for different reductions (R) factors. A total of 30 different structural models and 20 ground motions were used. The results showed that the current R factors provide conservative designs for low-rise steel buildings but showed a low level of confidence for high buildings. Krishnan et al. (2006) determined the damage produced by hypothetical earthquakes on two 18-storey steel MRF, one existing and one improved according to the 1997 Uniform Building Code (UBC), located in southern California, USA. They concluded that severe damage could occur in these buildings. The redesigned building performed significantly better than the existing one, however, the design based on the 1997 UBC was still not adequate to prevent serious damage. Liao et al. (2007) developed a three-dimensional finite-element model to examine the effects of bi-axial motion and torsion on the nonlinear response of steel MRF. Effects of gravity frames, panel zones, and inelastic column deformation were considered. Results indicated that torsional effects due to asymmetric member failures are important, that the conventional lumped-plasticity model limits the plasticity of columns and that fracture failures of the pre-Northridge connections have a severe impact on the buildings performance. Kazantzy et al. (2008) proposed a methodology for the probabilistic assessment of low-rise steel buildings and applied it to a welded MRF, emphasizing the modeling of connections. They found that structures experiencing connection brittle failure undergo large deformations, resulting in a low reliability in terms of achieving code-related performance parameters. More recently, Chang et al. (2009), by using 6- and 20-level steel office buildings, studied the role of accidental torsion in seismic reliability assessment. They concluded that ignoring the accidental torsion can lead to an unsafe evaluation of the strength of the building and that, on the other hand, the use of code accidental eccentricity may give conservative estimates.

Despite the large amount of research developed in the area of seismic behavior of steel buildings with MRF and the important contributions of the above-mentioned studies, and many others, a few studies have been developed in relation to the performance evaluation of steel buildings with MRF with deep columns. Shen et al. (2002) investigated the use of MRF with deep columns. They studied the seismic behavior of the 10-level model used in the SAC Steel Project (SAC, 1996) and developed an equivalent model replacing the W14 columns by W27 columns in order to compare their seismic behavior. Step-by-step nonlinear seismic analysis in time domain and incremental lateral static analysis (pushover) were performed. It was shown that using deep columns instead of W14 sections could result in a better behavior to resist lateral loads, much better control of drifts and damage or in reduced costs of construction. They showed that with the presence of the floor slab and the transverse beams is enough to eliminate or reduce the deep columns twisting to negligible levels without any consequences on the structure. It is important to mention that this study was limited to plane frames. Zhang et al. (2004) studied the seismic behavior of moment connections with reduced beam sections and deep wide flange columns, concluding that all specimens under study satisfied the criteria of the AISC seismic provisions and that the floor slab effect is very important since this can significantly reduce the lateral displacement of the bottom flange in the reduced beam section. Shao and Hale (2004) tested three full-scale beam-columns assemblies using W36x256 beams and W30x261 columns. They concluded that the proposed connections satisfy the two interstory displacement cycles of 0.04 and the inelastic rotation of 0.03 required for the Office of Satatewide Health Planning and Development (OSHPD).

3. OBJECTIVES

The main objective of this research is to compare the seismic responses of steel buildings with deep (W27) columns with the corresponding responses of buildings with columns of medium size (W14). Two of the steel building models considered in the SAC Steel Project (FEMA, 2000) are particularly studied. In these models, W14 columns at the PMRF are used. The W27 columns are selected according to two criteria: (1) equivalence in terms of strength, wherein the plastic moments about the major axis are approximately the same for the two types of sections (the W14 columns weight is approximately 60% higher and therefore represents a higher cost), and (2) equivalence in terms of weight (equal cost). The comparison is made in terms of single global (interstory displacements and shears), single local (axial loads and bending moments) and multiple local response parameters. Plane and three-dimensional models as well as elastic and inelastic analysis are considered. The structural models are submitted to the action of twenty seismic records, which have been previously selected taking into account their intensity, frequency contents and strong phase duration. For the inelastic deformation level the earthquakes are scaled so that the models suffer significant yielding but without producing their failure.

4. MATHEMATICAL MODEL

An assumed stress-based finite element algorithm (Gao and Haldar, 1995; Reyes-Salazar, 1997), is used to estimate the nonlinear seismic responses of the building models under consideration as accurately as possible. The procedure estimates the responses by considering the main sources of energy dissipation and 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. A computer program has been developed to implement the algorithm. The computer program has been extensively verified using information available in the literature. The structural responses in terms of member forces (axial and shear forces, and bending and torsional moments), interstory shears and displacements or any other response parameter, can be estimated using the program.

5. STRUCTURAL MODELS

Several steel model buildings with MRF were considered in the SAC steel project (FEMA, 2000). The models were designed by three consulting firms of United States according to the specifications of the following three cities codes: Los Angeles (Uniform Building (UBC 1994)), Seattle (UBC 1994) and Boston (Building Officials & Code Administration (BOCA 1993)). The 3- and 10-level buildings located in the Los Angeles area are considered in this study. They will be denoted hereafter as Models 1 and 2, respectively. The fundamental periods of Model 1 are estimated to be 1.03, 0.99 and 0.07 sec., in the *X* (horizontal), *Y* (horizontal) and *Z* (vertical) directions respectively.

The corresponding values for Model 2 are 2.22, 2.11 and 0.16 sec. The damping is considered to be 5% of the critical damping; the value used commonly in code provisions. The elevations of the models are given in Figs. 1(a) and 1(d) and their plans in Figs. 1(b) and 1(e). In these figures, the perimeters MRF are represented by continuous lines and the interior GF by dashed lines. Resultant forces are estimated for some particular columns, which are located at the ground floor level and are shown in Figs. 1(c) and 1(f) for Models 1 and 2, respectively.

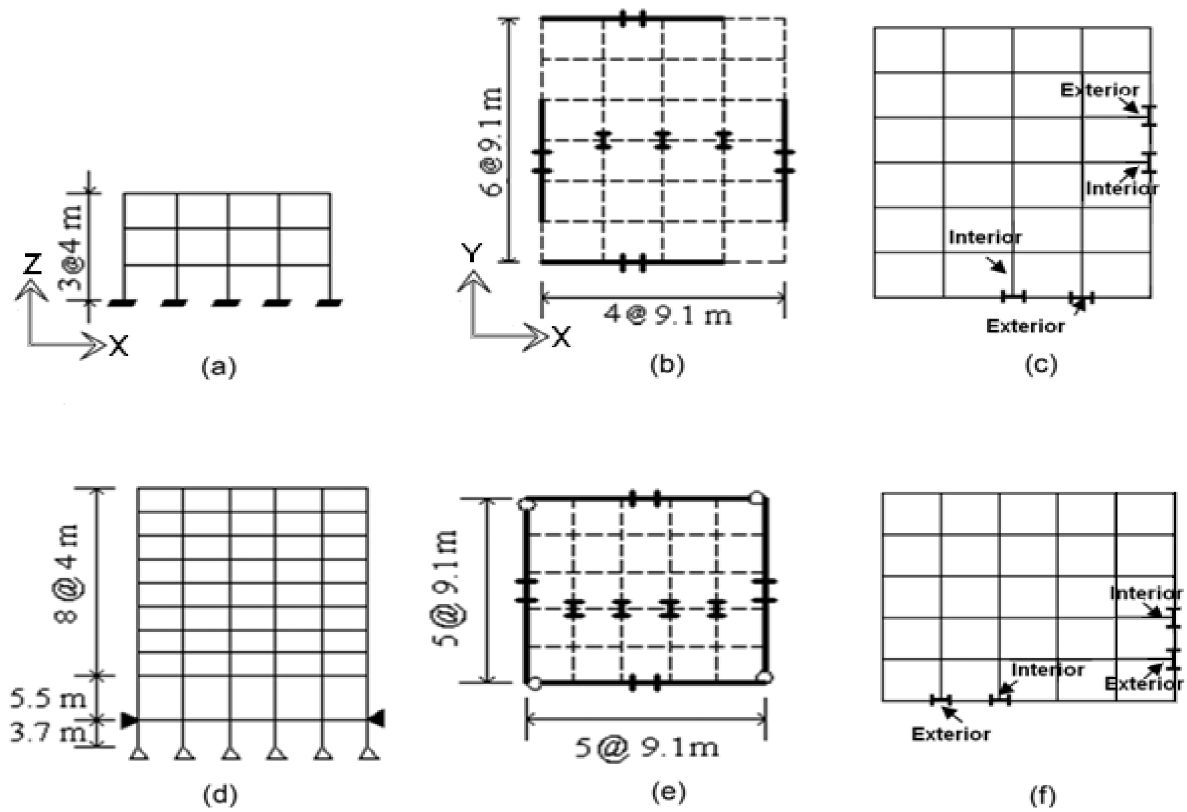


Fig. 1. (a) and (b) elevation and plan for Model 1, (d) and (e) elevation and plan for Model 2, (c) and (f) studied elements for Models 1 and 2

The columns of the perimeters MRF are of section W14; Table 1 shows the sections. As previously mentioned, the seismic response of these models is compared with the response of equivalent models (in terms of strength and weight) with deep columns. The column sections of the equivalent models are given in Table 2. The sections of the interior gravity frames and the beams of the perimeter MRF are the same for the SAC and equivalent models. The fundamental periods of the 3-level model in the *X* (horizontal), *Y* (horizontal) and *Z* (vertical) directions for the case of equivalent resistance are estimated to be 0.95, 0.91, and 0.08 sec., respectively, while the corresponding periods for the 10-level model are 2.08, 2.06 and 0.17 sec. For the equivalent weight the periods are 0.87, 0.85, and 0.07 sec. for the 3-level model and 1.93, 1.90 and 0.16 for the 10-level model. Comparing the W27 columns sections of the equivalent MRF in terms of weight with those of the SAC models (Tables 1 and 2) it is clearly observed that the weights are practically the same.

Table 1. Beam and column sections of SAC Models

MODEL	MOMENT RESISTING FRAMES				GRAVITY FRAMES		
	STORY	COLUMNS		GIRDERS	COLUMNS		GIRDERS
		EXTERIOR	INTERIOR		BELOW PENTHOUSE	OTHERS	
1	1/2	W14x257	W14x311	W33x118	W14x82	W14x68	W18x35
	2/3	W14x257	W14x311	W30x116	W14x82	W14x68	W18x35
	3/Roof	W14x257	W14x311	W24x68	W14x82	W14x68	W16x26
2	-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

As mentioned earlier, the models are excited by twenty seismic records, which are given in Table 3 and are denoted as Earthquakes 1 to 20. Their predominant periods, in terms of pseudo-acceleration, vary from 0.11 to 1.0 sec. and were selected in such way that the maximum accelerations of the horizontal components were at least *0.15 g* with a duration of the strong phase of at least 15 sec. The earthquake time histories were obtained from the Data Sets of the National Strong Motion Program (NSMP) of the United States Geological Surveys (USGS). The normal and principal components are used in the study. The components recorded directly by the measuring devices (seismograph) are defined as normal components. When such components are transformed to uncorrelated components the principal components are obtained.

6. LIMIT WIDH/THICK RATIO IN DEEP COLUMNS

The use of deep columns allows more easily achieving the strong-column weak-beam design requirements. However, their moments of inertia about the weak axis are relatively small in such a way that revision of the lateral torsional buckling limit state is required. In the Specifications of the American Institute of Steel Construction (AISC 2005a), in Chapter *F (Design of Members for Flexure)*, it is indicated that if the lateral unbraced length (L_b) of the compression flange of a compact I-shaped beam in flexion is smaller than an amount defined as L_p, lateral torsional buckling will not occur before the beam reaches its plastic moment capacity. In other words:

Table 2. Deep column sections for equivalent models

MODEL	EQUIVALENT STRENGTH			EQUIVALENT WEIGHT	
	STORY	COLUMNS		COLUMNS	
		EXTERIOR	INTERIOR	EXTERIOR	INTERIOR
1	1/2	W27x161	W27x194	W27x258	W27x307
	2/3	W27x161	W27x194	W27x258	W27x307
	3/Roof	W27x161	W27x194	W27x258	W27x307
2	-1/1	W27x217	W27x307	W27x368	W27x494
	1/2	W27x217	W27x307	W27x368	W27x494
	2/3	W27x217	W27x307,W27x281	W27x368	W27x494,W27x448
	3/4	W27x217	W27x281	W27x368	W27x448
	4/5	W27x217,W27x178	W27x281,W27x217	W27x368,W27x281	W27x448,W27x368
	5/6	W27x178	W27x217	W27x281	W27x368
	6/7	W27x178,W27x161	W27x217,W27x178	W27x281,W27x258	W27x368,W27x281
	7/8	W27x161	W27x178	W27x258	W27x281
	8/9	W27x161,W27x146	W27x178,W27x161	W27x258,W27x235	W27x281,W27x258
	9/Roof	W27x146	W27x161	W27x235	W27x258

Table 3. Earthquake Models

EARTH.	PLACE	YEAR	STATION	T (sec.)	EPICENTER (Km.)	DEPTH (Km.)	MAG.	PGA mm/s^2
1	1317 Mich. México	1985	Paraíso	0.1	300	15	8.1	800
2	1634 Mammoth Lakes.	1980	Mammoth H. S. Gym	0.1	11	9	6.5	2000
3	1634 Mammoth Lakes	1980	Convict Creek	0.2	8	9	6.5	3000
4	1317 Mich. México	1985	Infiernillo N-120	0.2	67	28	8.1	3000
5	1317 Mich. México	1985	La Unión	0.3	121	15	8.1	1656
6	1733 El Salvador	2001	Relaciones Ext.	0.3	96	60	7.8	2500
7	1733 El Salvador	2001	Relaciones Ext.	0.4	95	60	7.8	1500
8	1634 Mammoth Lakes.	1980	Long Valley Dam	0.4	13	9	6.5	2000
9	2212 Delani Fault, AK	2000	K2-02	0.4	281	5	7.9	115
10	0836 Yountville CA	2000	Redwood City	0.46	95	9	5.2	90
11	0408 Dillon MT	2005	MT:Kalispell	0.5	338	5	5.6	51
12	1317 Mich. Mexico	1985	Villita	0.5	80	15	8.1	1225
13	1232 Northrige	1994	Hall Valley	0.5	25	15	6.4	2500
14	2115 Morgan Hill	1984	Hall Valley	0.6	14	8	6.2	2000
15	2212 Delani Fault AK	2002	K2-04	0.6	290	5	7.9	133
16	0836 Yountville CA	2000	Dauville F.S. Ca	0.6	73	9	5.2	144
17	0836 Yountville CA	2000	Pleasan Hill F.S. 1	0.7	92	9	5.2	74
18	0836 Yountville CA	2000	Pleasan Hill F.S. 2	0.7	58	9	5.2	201
19	2212 Delani Fault, AK	2002	Valdez City Hall	0.8	272	5	7.9	260
20	1715 Park Fiel	2004	CA: Hollister City Hall	1	14	8	6	145

if $L_b \le L_p$, the limit state of lateral torsional buckling does not apply and,

$$M_n = M_p = F_y\, Z_x \tag{1}$$

where,

$$L_p = 1.76\, r_y \sqrt{\frac{E}{F\mathrm{y}}} \tag{2}$$

M_n = Nominal strength in bending,
M_p = Plastic moment,
F_y = Yield stress specified for the type of steel used,
Z_x = Plastic section modulus about the strong axis (axis X),
r_y = Radius of gyration about the weak axis,
E = Elasticity modulus of steel.

Besides, if the width/thick ratio of the elements that form the cross sections of I-shaped columns ($b_f/2t_f$ for flange and h/t_w for the web) does not exceed a limit value, local buckling of such elements will not occur before buckling member. The limit relations, for the case of compact columns are given by the following expressions:

$$\frac{b_f}{2t_f} = 0.38\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} \tag{3}$$

$$\frac{}{t_w} = 3.76\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} \tag{4}$$

Additionally, for the case of deep columns, where the web is relatively slender, shear buckling of panel zone may occur. This failure mode can be avoided by limiting the relation h/t_w of the column web to the value given by the following expression (AISC 2005a):

$$\frac{}{t_w} \leq 2.24\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} \tag{5}$$

If the h/t_w ratio of the column web satisfies Eq. (5), it is expected that the column web can reach yielding by shear before buckling, and the nominal shear strength (V_n), could be calculated with by:

$$V_n = 0.6F_yA_w \tag{6}$$

where A_w is the web area. From an observation of the properties and dimensions of the *W* column sections available in the tables of the AISC manual (AISC 2005b), it is found that the W27 sections Grade 50, which are used as deep columns in this study, satisfy Eqs. (3), (4) and (5).

7. 3D MODELS WITH EQUIVALENT COLUMNS IN TERMS OF STRENGTH

In this chapter, the seismic responses of buildings with W14 columns, modeled as three-dimensional structures, are compared with the corresponding responses of buildings with W27 columns. To compare the interstory displacements the parameter D_1, defined as $D_{3,M,R}/D_{3,L,R}$, is used. It must be noted that the subscripts M, L, and R abbreviate the words *medium*, *large* and *resistance*, which in turn refer to *medium column size*, *large column size* and *equivalent resistance (or strength)*, respectively. For a given model, direction and interstory, $D_{3,M,R}$ represents the average interstory displacements of all frames in that direction when W14 columns are used in the PMRF, while $D_{3,L,R}$ represents the same but W27 columns are used instead. Typical values of D_1 are presented in Figs. 2a and 2b for the 3- and 10-level models, respectively, for the case of normal components, elastic behavior and X direction. The symbol *ET* is used in the figure to represent the word *interstory*. It is observed that the D_1 values significantly vary from one earthquake to another and from one story to another without showing any trend, and that these values are generally greater than unity, reaching values of up to 1.5. The implication of this is that the displacements are larger for models with W14 columns. Similar ratios for some individual frames were also calculated. However, the results are not presented because there were not significant differences.

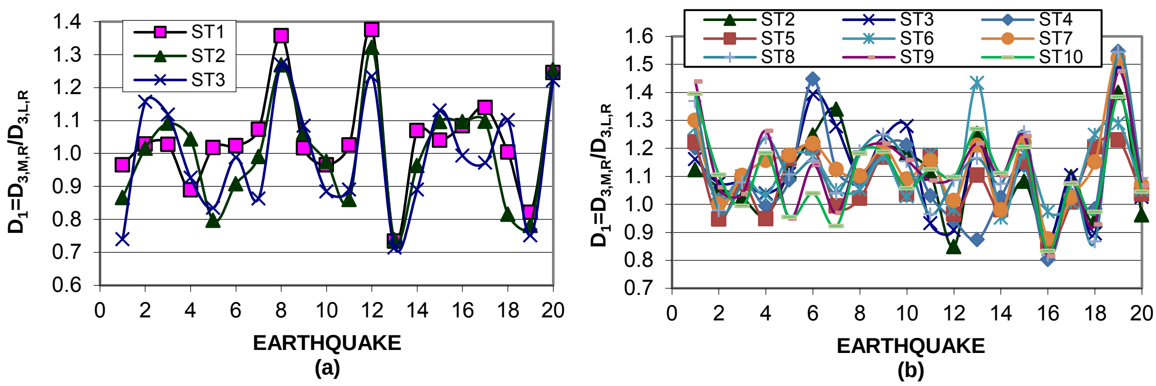


Fig. 2. D_1 values, normal components, elastic, X direction, (a) Model 1 and (b) Model 2

The V_1 parameter, defined as $V_{3,M,R}/V_{3,L,R}$, is used to compare the interstory shears. This parameter has a similar meaning that D_1 but average interstory shears and interstory shears for individual frames are considered in this case. The average V_1 values for the 3- and 10-level models are presented in Figs. 3a and 3b, respectively, for the case of normal components, elastic behavior and X direction. The V_1 values for the 3-level model are presented in Figs. 4a and 4b for an exterior and an interior frame, respectively, for the case of normal components, elastic behavior and X direction too. The results indicate that for average shears and shears at exterior frames, the V_1 values are similar and generally less than unity, values smaller than 0.7 are observed for some cases, implying that the shears are, in general, higher for models with deep columns. Like D_1, V_1 significantly vary from one earthquake to another and from one story to another without showing any trend. As expected, a large correlation is observed between D_1 and V_1. For interior frames (Fig. 4b), the shear ratios are considerably greater than unity; for the case presented, they practically vary from 1.4 to 2.6, indicating that the shears are greater for the models with W14 columns.

The previous figures represent typical values of the D_1 and V_1 parameters. However, considering two models, two directions, normal and principal components, elastic and inelastic behavior, and displacements and shears (average and individuals) as global response parameters, a total of 64 figures were developed, which are not presented because of lack of space. Only the fundamental statistics in terms of mean (μ) and coefficient of variation (δ) are presented for all cases. The results are given in Table 4.

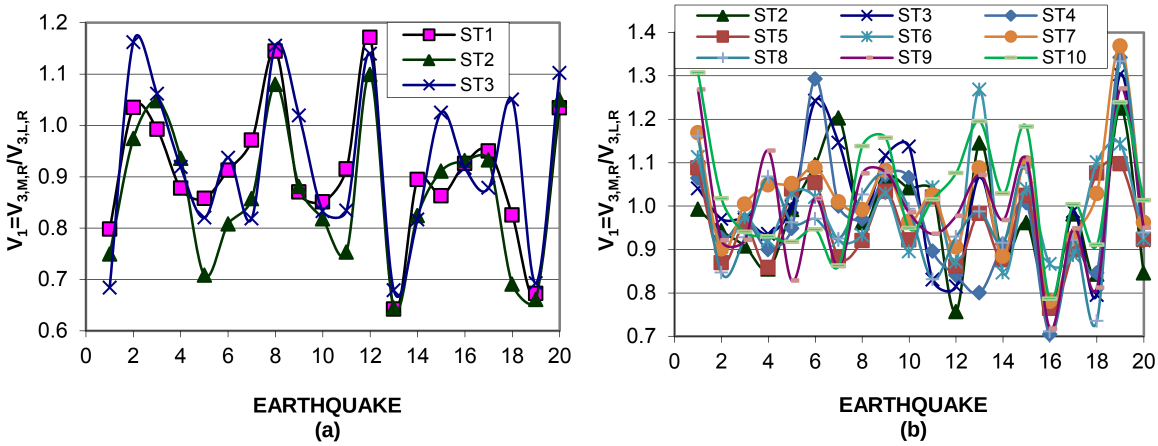


Fig. 3. V_1 values, normal components, elastic, *X* direction, (a) Model 1 and (b) Model 2

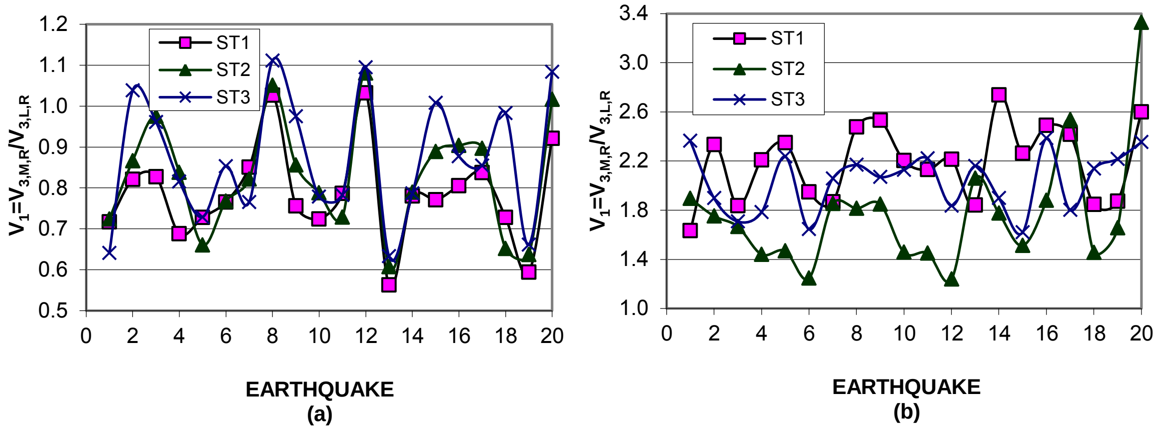


Fig. 4. V_1 values for individual frames, Model 1, normal components, elastic, *X* direction, (a) exterior frame and (b) interior frame

Table 4 corroborates what observed from individual plots: in terms of averages, the D_1 values indicate that the displacements are generally larger for models with W14 columns and the V_1 values that the shears are larger for models with W27 columns. However, the differences between the results of the two columns sizes are greater for the case of displacements; D_1 mean values close to 1.20 are observed in several cases while those of V_1 are close to unity in most cases. The uncertainty in the estimation is similar for both D_1 and V_1, ranging from 9 to 18%, indicating a moderate dispersion. For individual frames, the V_1 mean values are generally smaller than unity for exterior frames but significantly greater than unity for interior frames, values larger than 2 are observed in many cases, the maximum value observed is 2.75. The implication of this is that interstory shears at the interior GF is significantly reduced when deep columns are used. This helps to counteract the no conservative effect that occurs in the design practice of the structural system under consideration, when lateral seismic forces are not considered in the design of GF. It is also observed that the uncertainty in the estimation of V_1 is greater for interior frames. Significant differences are not observed between the statistics of normal and principal components, or elastic and inelastic behavior.

Table 4. Statistic for the D_1 and V_1 parameter

MODEL	BEHAVIOR	STORY	**D_1 (AVERAGE)**				**V_1 (AVERAGE)**				**V_1 (INDIVIDUAL FRAMES)**							
											EXTERIOR FRAME				**INTERIOR FRAME**			
			DIRECTION				**DIRECTION**				**DIRECTION**				**DIRECTION**			
			X		Y		X		Y		X		Y		X		Y	
			μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC, NORMAL	1	1.05	15	1.09	15	0.91	14	0.89	15	0.79	15	0.82	15	2.19	14	2.00	16
		2	1.00	17	1.09	15	0.87	16	0.91	15	0.83	17	0.89	15	1.77	27	1.29	12
		3	0.99	17	1.07	18	0.93	17	0.97	18	0.87	18	0.94	19	2.03	12	1.54	14
	INELASTIC, NORMAL	1	1.06	15	1.02	12	0.92	14	0.90	15	0.80	14	0.83	16	2.19	14	1.91	16
		2	1.01	16	1.09	16	0.87	16	0.93	14	0.83	16	0.93	14	1.81	32	1.11	24
		3	0.99	17	1.08	17	0.93	16	1.00	17	0.88	17	0.97	18	2.02	13	1.36	14
	ELASTIC, PRINCIPAL	1	1.01	16	1.18	16	0.87	16	0.97	17	0.75	16	0.89	17	2.12	17	2.14	16
		2	1.00	14	1.13	14	0.86	14	0.95	14	0.82	14	0.93	14	1.77	25	1.34	16
		3	0.98	16	1.08	16	0.92	15	0.98	16	0.87	16	0.95	16	1.98	10	1.60	15
	INELASTIC, PRINCIPAL	1	1.01	16	1.09	18	0.87	16	0.99	15	0.76	17	0.91	15	2.11	16	2.03	18
		2	1.01	14	1.11	13	0.87	15	0.97	15	0.83	15	0.95	16	1.75	23	1.18	22
		3	0.99	16	1.08	15	0.93	15	0.99	15	0.88	17	0.97	17	1.99	13	1.34	19
2	ELASTIC, NORMAL	2	1.10	13	1.06	13	0.98	14	0.95	13	0.89	13	0.86	13	1.72	14	1.38	13
		3	1.12	15	1.07	13	1.00	14	0.95	13	0.98	15	0.94	13	1.28	12	1.17	13
		4	1.09	16	1.05	14	0.97	15	0.93	13	0.96	16	0.91	13	2.08	22	1.55	14
		5	1.07	10	1.06	13	0.96	10	0.95	13	0.95	10	0.94	14	2.05	9	1.48	12
		6	1.12	11	1.11	11	0.99	11	0.98	11	0.97	11	0.96	11	1.97	11	1.53	10
		7	1.14	12	1.11	13	1.02	12	1.00	13	1.00	12	0.98	13	2.07	13	1.64	11
		8	1.13	15	1.12	16	0.97	15	0.96	16	0.95	15	0.94	16	2.73	15	2.04	19
		9	1.13	14	1.12	16	0.99	14	0.98	17	0.97	14	0.96	17	2.05	11	1.49	15
		10	1.11	13	1.11	17	1.03	13	1.04	16	1.01	14	1.02	17	2.65	9	1.77	11
	INELASTIC, NORMAL	2	1.1	1	1.0	1	0.9	1	0.9	1	0.89	13	0.89	14	1.7	1	1.3	1

			0	3	4	2	8	3	7	3					3	4	4	4
		3	1.12	14	1.06	13	1.00	14	0.98	12	0.99	14	0.97	12	1.29	12	1.04	22
		4	1.10	16	1.05	14	0.97	16	0.95	13	0.96	16	0.94	14	2.05	21	1.33	25
		5	1.08	10	1.07	14	0.96	10	0.97	13	0.95	10	0.96	14	2.03	13	1.22	31
		6	1.11	12	1.11	11	0.99	11	1.00	11	0.97	11	0.99	11	1.96	12	1.23	26
		7	1.13	12	1.12	13	1.02	12	1.02	12	1.01	12	1.01	13	2.13	13	1.39	22
		8	1.12	15	1.12	17	0.97	15	0.97	16	0.95	15	0.95	16	2.63	16	1.67	30
		9	1.13	14	1.13	17	0.99	15	0.99	16	0.97	15	0.97	17	2.05	10	1.42	17
		10	1.10	13	1.13	17	1.03	13	1.06	15	1.00	13	1.03	16	2.64	10	1.76	13
	ELASTIC, PRINCIPAL	2	1.10	12	1.02	14	0.98	12	0.91	14	0.90	12	0.83	14	1.73	13	1.33	14
		3	1.13	16	1.05	15	1.01	16	0.94	15	1.00	16	0.93	15	1.26	12	1.17	17
		4	1.08	15	1.05	12	0.96	15	0.93	12	0.95	15	0.92	12	2.07	22	1.54	12
		5	1.09	13	1.06	10	0.97	13	0.95	10	0.96	13	0.94	11	2.02	11	1.55	12
		6	1.13	12	1.08	9	1.00	12	0.96	9	0.98	12	0.94	10	1.96	13	1.50	12
		7	1.14	10	1.08	13	1.03	10	0.97	12	1.01	10	0.95	13	2.11	12	1.59	10
		8	1.14	14	1.11	15	0.98	14	0.95	14	0.96	14	0.93	15	2.75	16	2.01	18
		9	1.11	14	1.11	16	0.98	13	0.98	16	0.96	14	0.96	16	2.04	12	1.49	12
		10	1.11	13	1.09	16	1.03	12	1.02	15	1.01	13	0.99	16	2.67	12	1.75	8
	INELASTIC, PRINCIPAL	2	1.11	12	0.99	14	0.99	12	0.95	14	0.90	11	0.88	15	1.76	13	1.26	14
		3	1.13	16	1.03	16	1.01	15	0.96	14	1.00	15	0.96	15	1.29	12	0.98	26
		4	1.08	16	1.05	14	0.96	15	0.96	12	0.95	15	0.95	13	1.98	23	1.28	23
		5	1.09	13	1.07	11	0.98	12	0.97	11	0.97	12	0.97	12	1.95	16	1.20	30
		6	1.13	12	1.08	9	1.00	11	0.99	10	0.99	11	0.98	11	1.96	14	1.26	23
		7	1.14	10	1.09	12	1.03	10	1.00	12	1.01	10	0.99	12	2.09	13	1.27	23
		8	1.14	14	1.10	16	0.98	13	0.99	13	0.96	14	0.98	14	2.67	16	1.51	34
		9	1.12	13	1.12	16	0.99	13	1.01	14	0.96	13	0.99	15	2.02	13	1.41	18
		10	1.11	12	1.11	16	1.03	12	1.04	14	1.00	12	1.01	15	2.61	13	1.65	19

The responses in terms of local parameters (axial force and bending moment) are now discussed for some particular columns which are located at the ground floor level (Figs. 1c and 1f). The ratios $A_1 = A_{3,M,R}/A_{3,L,R}$ and $M_1 = M_{3,M,R}/M_{3,L,R}$, are used to compare axial forces and bending moments, respectively, between the two column sizes. The A_1 and M_1 values are given in Figs. 5(a) and 5(b), for the 3-level model, Y direction, principal components and inelastic behavior. As for the global parameters previously discussed, the A_1 values vary from one earthquake to another and from one element to another without showing any trend. The M_1 values present a high correlation between interior and exterior columns. It is observed that the A_1 values may be greater or smaller than unity while those of M_1 are always smaller than unity.

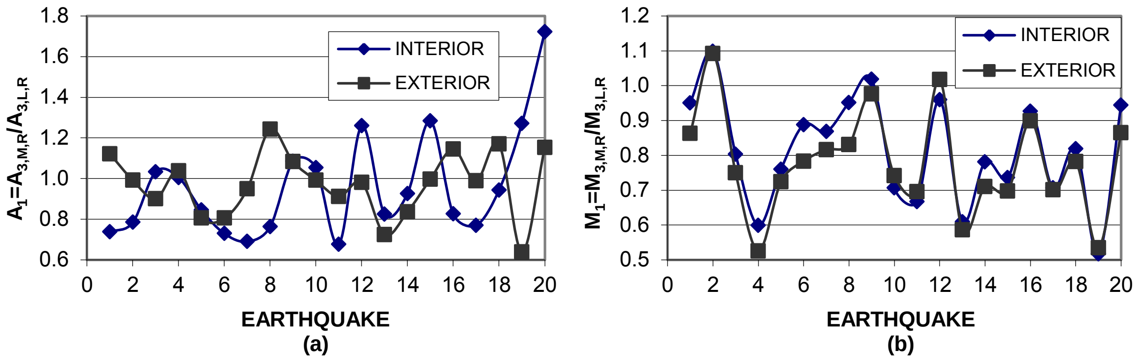


Fig. 5. (a) A_1 values and (b) M_1 values; Model 1, principal components, inelastic, *Y* direction

The statistics of A_1 and M_1 are shown in Table 5. The A_1 mean values are larger for the case of elastic behavior and interior columns, the maximum value observed is 1.58. In the remaining cases the axial forces are generally larger when W27 columns are used. The uncertainty in the estimation of A_1 is considerable, δ values greater than 40% are found in some cases. It is observed that the M_1 mean values are smaller than unity in all cases, the minimum observed value is of 0.65. They are larger for 10-level Model and interior columns. The uncertainty in the estimation of A_1 is larger than that of M_1. Significant differences are not observed between the statistics of elastic and inelastic behavior or normal and principal components.

Table 5. Statistics for the A_1 and M_1 parameters

MODEL		MEMBER LOCATION	***A₁***								***M₁***							
			NORMAL COMPONENT				PRINCIPAL COMPONENT				NORMAL COMPONENT				PRINCIPAL COMPONENT			
			X DIRECTION		Y DIRECTION		X DIRECTION		Y DIRECTION		X DIRECTION		Y DIRECTION		X DIRECTION		Y DIRECTION	
			μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC	INTERIOR	1.58	40	1.56	39	1.56	34	1.47	26	0.71	15	0.73	15	0.68	16	0.80	17
		EXTERIOR	0.98	19	1.04	20	0.97	20	1.11	20	0.68	15	0.70	16	0.65	16	0.76	17
	INELASTIC	INTERIOR	1.10	25	0.97	25	1.06	43	0.96	27	0.71	14	0.74	21	0.68	18	0.82	19
		EXTERIOR	0.91	17	0.98	17	0.90	18	0.97	16	0.69	16	0.72	21	0.65	17	0.78	19
2	ELASTIC	INTERIOR	1.32	44	1.28	27	1.33	44	1.36	32	0.86	13	0.83	13	0.86	12	0.80	15
		EXTERIOR	0.97	10	1.02	17	0.97	14	1.04	18	0.83	13	0.80	13	0.84	12	0.77	14
	INELASTIC	INTERIOR	0.88	20	0.90	20	0.90	16	0.89	22	0.86	13	0.85	18	0.87	12	0.85	18
		EXTERIOR	0.96	9	0.99	11	0.97	8	0.99	10	0.83	13	0.83	18	0.84	12	0.82	18

8. 2D MODELS WITH EQUIVALENT COLUMNS IN TERMS OF STRENGTH

The seismic responses of buildings with columns of medium and large depth, modeled as two-dimensional structures, are presented in this section of the paper. The D_2 parameter, defined as $D_{2,M,R}/D_{2,L,R}$, is used to compare the interstory displacements of the buildings with the two sizes of columns. $D_{2,M,R}$ and $D_{2,L,R}$ represent the interstory displacement when medium and large columns are considered, respectively. Similarly, the V_2 parameter, calculated as $V_{2,M,R}/V_{2,L,R}$, is used to compare the interstory shears. V_2 is similar in definition to D_2 with the difference that interstory shears are now considered. As for the 3D case, plots of these parameters were developed for the 3- and 10-level models, two directions, normal and principal components, and elastic and inelastic deformation. The D_2 and V_2 values for the 3-level model, are presented in Figs. 6(a) and 6(b), respectively, for the case of normal components, inelastic behavior and X direction. It can be seen that the displacements ratios (D_2) are generally greater than unity, values close to 1.8 are observed for Story 1. The V_2 values are smaller than unity in most of the cases.

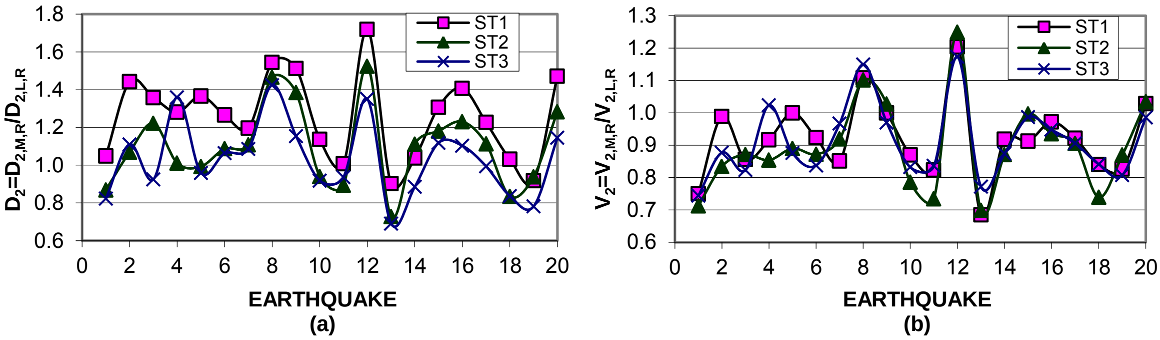


Fig. 6. (a) D_2 values and (b) V_2 values, Model 1, normal components, inelastic, X direction

Table 6 shows the statistics of D_2 and V_2. They confirm what was observed from individual figures: displacements are larger for the models with W14 columns while shears are slightly larger for the models with W27 columns. As for the 3D case, the uncertainty in the estimation of D_2 and V_2 is not considerable. The statistical values of D_2 and V_2 are quite similar for elastic and inelastic behavior and for normal and principal components. By comparing the mean values of the displacements ratios of the 2D (D_2) and 3D (D_2) modeling, it is observed that they are similar in the sense that the ratios are, in general, greater than unity. However, they are larger for the 2D models, indicating a greater difference between the displacements of the buildings with W14 and W27 columns. The implication of this is that the seismic responses of the buildings under consideration modeled as 2D structures may be different than those of the 3D modeling. In the case of shear ratios no significant differences are observed between the 2D and 3D modeling.

The A_2 and M_2 ratios, defined as $A_{2,M,R}/A_{2,L,R}$ and $M_{2,M,R}/M_{2,L,R}$, are used to compare axial forces and bending moments, respectively, of the buildings with the two column sizes. Several plots for these two parameters were also developed, but the results are not presented. It is observed from the plots, however, that the A_2 values are, in most of the cases, smaller than unity. For some particular cases, however, they are significantly greater than unity. The statistics for all the cases are shown in Table 7. The largest value of the mean of A_2 is observed for interior columns of the 3-level buildings, for the case of elastic behavior and normal components. The individual and mean values of M_2 are practically smaller than unity in all cases. The mean values are larger for the 10-

level model. The M_2 values are, in general, lower than the A_2 values. It is observed that the mean values and the uncertainty in the estimation of the axial load ratios are larger for the 3D (A_1) than for the 2D modeling (A_2) while for the case of bending moment ratios they are larger for the 2D (A_2) than for the 3D (A_1) modeling.

Table 6. Statistics for the D_2 and V_2 parameters

MODEL		STORY	D_2				V_2			
			NORMAL COMPONENT		PRINCIPAL COMPONENT		NORMAL COMPONENT		PRINCIPAL COMPONENT	
			μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC	1	1.25	22	1.20	23	0.89	21	0.86	22
		2	1.07	23	1.03	22	0.87	23	0.85	22
		3	1.02	23	1.00	26	0.90	23	0.88	26
	INELASTIC	1	1.26	18	1.19	18	0.92	13	0.88	15
		2	1.10	19	1.07	18	0.90	15	0.90	16
		3	1.03	19	1.02	25	0.91	13	0.90	17
2	ELASTIC	2	1.18	16	1.22	16	0.95	15	0.98	16
		3	1.06	17	1.09	16	0.93	16	0.96	16
		4	1.06	15	1.07	14	0.92	15	0.94	14
		5	1.07	13	1.10	15	0.94	13	0.96	15
		6	1.11	16	1.15	17	0.96	15	1.00	17
		7	1.08	17	1.12	14	0.95	17	0.99	15
		8	1.10	20	1.13	17	0.93	19	0.95	16
		9	1.11	19	1.12	18	0.95	18	0.95	18
		10	1.07	18	1.07	17	0.95	16	0.96	16
	INELASTIC	2	1.17	15	1.22	15	0.94	15	0.97	14
		3	1.06	16	1.09	16	0.94	15	0.97	14
		4	1.05	15	1.07	15	0.92	15	0.95	12
		5	1.06	13	1.09	16	0.94	12	0.96	14
		6	1.10	16	1.14	17	0.96	15	0.99	16
		7	1.08	17	1.11	15	0.96	17	0.99	15
		8	1.10	20	1.12	16	0.93	18	0.96	15
		9	1.11	19	1.13	18	0.95	17	0.96	16
		10	1.07	17	1.07	18	0.95	15	0.96	13

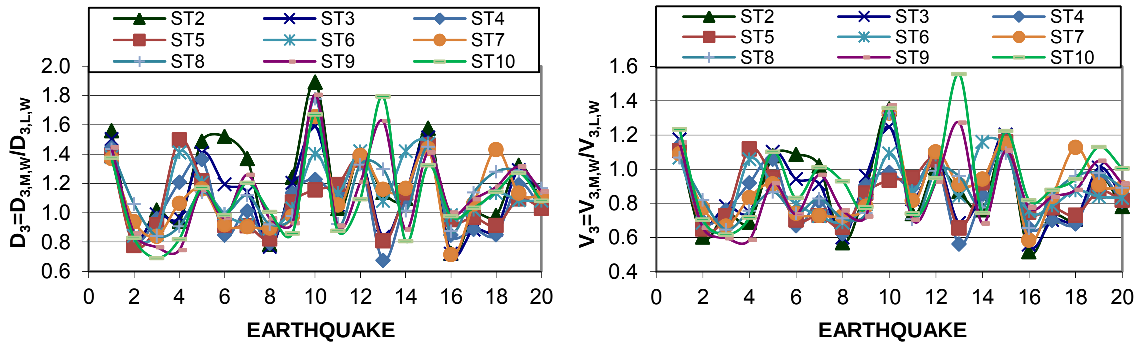
9. 3D MODELS WITH EQUIVALENT COLUMNS IN TERMS OF WEIGHT

The seismic responses of 3D buildings with W14 columns are now compared with those of 3D buildings with equivalent W27 columns in terms of weight. Similar parameters to those of the case of equivalent strength are defined for this purpose. The $D_3 = D_{3,M,W}/D_{3,L,W}$ and $V_3 = V_{3,M,W}/V_{3,L,W}$ parameters, are used to compare the interstory displacements and shears, respectively. The terms in the above expressions have the same meaning as before, the only difference is that now, as previously commented, the equivalence between columns is given in terms of weight. The D_3 and V_3 values are shown in Figs. 7(a) and 7(b), respectively, for the 10-level model, principal components, elastic behavior and the Y direction. It can be seen that the D_3 ratio considerably varies from one earthquake to another and from one story to another. However, the variation from one interstory to another is smaller. It is observed that, as for the equivalent strength case, the D_3 values are, in general, greater than unity, reaching values of up to

1.9, implying that the displacements can be significantly larger for the models with W14 columns. For the case of shear ratios, their values are generally smaller than unity, varying, practically from 0.6 to 1.4. Figures for displacements and shears for individual frames were also developed but are not presented. The observations are essentially the same as those of the equivalent strength case

Table 7. Statistics for the A_2 and M_2 parameters

MODEL		MEMBER LOCATION	A_2				M_2			
			NORMAL COMPONENT		PRINCIPAL COMPONENT		NORMAL COMPONENT		PRINCIPAL COMPONENT	
			μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC	INTERIOR	1.11	25	1.06	22	0.80	22	0.77	23
		EXTERIOR	1.06	28	1.02	28	0.78	22	0.75	23
	INELASTIC	INTERIOR	0.98	15	0.97	18	0.83	14	0.80	16
		EXTERIOR	1.01	20	0.98	16	0.79	14	0.78	16
2	ELASTIC	INTERIOR	1.00	34	0.99	31	0.94	16	0.97	15
		EXTERIOR	0.98	19	0.95	15	0.91	16	0.94	16
	INELASTIC	INTERIOR	0.92	22	0.89	12	0.93	14	0.96	15
		EXTERIOR	0.96	20	0.96	15	0.90	15	0.93	15



(a) (b)

Fig. 7. (a) D_3 values and (b) V_3 values, Model 2, principal components, elastic, *Y* direction

The Statistics for D_3 and V_3 are given in Table 8. It is observed that, for the case of averages, the D_3 mean values are greater than unity reaching values of up to 1.49 while the V_3 mean values are lower than unity in practically all the cases. Both parameters show considerable dispersion (close to 40%) for some cases of the 3-levels model. Significant differences are not observed between the statistics of normal and principal components or elastic and inelastic behavior. It is noted that, as in the case of interstory average shear, the V_3 mean values of exterior frames are lower than unity. On the other hand, the values for interior frames are much larger than unity, values close to 3 are observed in some cases. The reason for this is that the W27 columns attract higher interstory shears than the W14 columns, accordingly the interstory shears at the interior GF of models with W27 columns are much smaller than those of the models with W14 columns. As previously commented, this helps to reduce the no conservative effect implicitly considered in the design practice of steel building with PMRF when lateral seismic forces are not included in the interior GF. Comparing the means values of the average interstory displacement ratios found in this section (D_3) with those corresponding to the equivalence in terms of strength (D_1), it is observed that they are higher for the case of equivalence in terms of weight. This conclusion also applies to interstory displacements of individual frames. For average shears and shears at exterior individual frames, the mean values are generally smaller for equivalence in strength, but for interior individual frames they are larger for the case of equivalence in weight. For both shears and displacements, the uncertainty in the estimation is generally smaller for the equivalent strength case.

Table 8. Statistics for the D_3 and V_3 parameters

MODEL	BEHAVIOR	STORY	**D_3 (AVERAGE)**				**V_3 (AVERAGE)**				**V_3 (INDIVIDUAL FRAMES)**							
											EXTERIOR FRAME				**INTERIOR FRAME**			
			DIRECTION				**DIRECTION**				**DIRECTION**				**DIRECTION**			
			X		**Y**		**X**		**Y**		**X**		**Y**		**X**		**Y**	
			μ	δ	M	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC NORMAL	1	1.31	38	1.43	25	0.89	37	0.90	24	0.78	39	0.84	25	1.88	28	1.85	25
		2	1.15	33	1.29	26	0.85	33	0.92	25	0.82	34	0.90	26	1.44	30	1.36	22
		3	1.09	32	1.25	26	0.93	31	1.02	26	0.88	32	0.99	27	1.63	21	1.49	18
	INELASTIC NORMAL	1	1.31	37	1.28	24	0.88	33	0.89	25	0.77	35	0.83	26	1.85	27	1.70	22
		2	1.15	34	1.23	25	0.85	30	0.92	25	0.82	31	0.90	26	1.46	36	1.29	24
		3	1.08	29	1.22	26	0.93	29	0.97	23	0.89	30	0.95	25	1.60	23	1.36	27
	ELASTIC PRINCIPAL	1	1.28	38	1.49	23	0.86	38	0.94	23	0.76	40	0.87	23	1.83	31	1.91	25
		2	1.15	33	1.30	25	0.85	33	0.94	24	0.82	34	0.91	25	1.46	29	1.38	22
		3	1.09	32	1.17	27	0.91	30	0.96	27	0.87	32	0.93	28	1.66	21	1.49	22
	INELASTIC PRINCIPAL	1	1.28	39	1.42	27	0.85	35	0.93	23	0.74	37	0.88	23	1.76	29	1.80	29
		2	1.15	33	1.25	27	0.83	32	0.93	24	0.80	32	0.91	24	1.55	40	1.26	43
		3	1.10	32	1.17	28	0.90	29	0.95	25	0.85	31	0.93	26	1.70	22	1.32	27
2	ELASTIC NORMAL	2	1.30	26	1.19	29	0.93	26	0.86	28	0.84	26	0.77	28	1.73	25	1.43	27
		3	1.22	28	1.12	27	0.96	28	0.88	27	0.94	28	0.86	28	1.63	24	1.45	19
		4	1.18	24	1.05	25	0.93	24	0.83	24	0.91	24	0.82	25	1.92	19	1.74	21
		5	1.19	20	1.09	23	0.94	20	0.86	22	0.93	21	0.85	23	1.48	25	1.31	21
		6	1.29	20	1.18	20	0.99	19	0.91	19	0.97	20	0.89	20	2.44	22	1.93	22
		7	1.23	20	1.14	24	0.98	20	0.90	23	0.96	20	0.88	24	1.99	18	1.77	18
		8	1.22	21	1.12	25	0.92	21	0.85	25	0.90	22	0.83	25	2.89	30	2.40	29
		9	1.20	19	1.13	26	0.94	19	0.88	27	0.92	19	0.86	27	1.89	15	1.58	23
		10	1.13	17	1.12	26	0.97	19	0.98	27	0.95	19	0.95	28	2.54	23	2.02	19
	INELASTIC NORMAL	2	1.31	26	1.15	27	0.92	23	0.88	27	0.84	22	0.79	27	1.74	23	1.35	25
		3	1.24	29	1.11	26	0.96	24	0.89	25	0.94	24	0.88	25	1.63	24	1.23	20
		4	1.19	26	1.06	26	0.93	21	0.85	23	0.92	21	0.84	24	1.71	24	1.47	38
		5	1.19	21	1.09	24	0.95	18	0.88	21	0.94	18	0.88	22	1.41	33	1.16	34
		6	1.27	19	1.17	21	1.00	17	0.94	18	0.98	17	0.92	18	2.28	25	1.61	39
		7	1.20	20	1.13	26	0.99	17	0.93	22	0.97	17	0.91	22	2.12	21	1.52	30
		8	1.19	20	1.11	26	0.92	20	0.87	24	0.90	20	0.84	24	2.52	34	1.96	49
		9	1.19	17	1.13	26	0.94	18	0.90	25	0.92	18	0.88	26	1.98	21	1.48	25
		10	1.14	17	1.13	25	0.96	18	0.99	25	0.94	18	0.96	26	2.34	17	1.84	25
	ELASTIC PRINCIPAL	2	1.31	28	1.20	26	0.93	27	0.87	26	0.85	27	0.78	26	1.72	25	1.43	23
		3	1.22	28	1.12	24	0.95	28	0.88	23	0.94	28	0.87	24	1.59	23	1.44	16
		4	1.14	25	1.07	22	0.89	24	0.84	21	0.88	25	0.83	22	1.89	19	1.70	20
		5	1.14	19	1.08	20	0.90	18	0.86	19	0.90	19	0.85	20	1.53	27	1.33	19
		6	1.26	20	1.14	18	0.97	19	0.88	18	0.95	20	0.86	18	2.36	25	1.85	21
		7	1.25	21	1.12	21	0.98	20	0.89	21	0.97	21	0.87	21	2.07	19	1.73	19
		8	1.24	24	1.15	21	0.94	23	0.87	20	0.92	24	0.84	20	2.98	31	2.34	24
		9	1.18	22	1.13	25	0.91	22	0.88	25	0.89	23	0.86	26	1.84	16	1.55	22
		10	1.12	20	1.10	26	0.96	22	0.96	25	0.93	22	0.93	26	2.63	29	2.00	13
	INELASTIC	2	1.33	27	1.15	24	0.94	26	0.87	24	0.85	25	0.79	26	1.75	25	1.33	22

		PRINCIPAL	3	1.22	29	1.10	25	0.96	27	0.89	22	0.94	27	0.89	22	1.59	24	1.09	36
			4	1.14	26	1.06	24	0.90	23	0.86	21	0.89	24	0.85	21	1.62	27	1.33	36
			5	1.13	20	1.08	23	0.92	17	0.88	18	0.91	18	0.87	19	1.44	28	1.06	39
			6	1.25	20	1.14	21	0.98	19	0.90	17	0.97	19	0.88	17	2.06	31	1.47	44
			7	1.24	21	1.12	22	0.99	19	0.90	19	0.98	20	0.89	19	2.09	16	1.28	33
			8	1.24	23	1.12	21	0.94	22	0.88	19	0.92	23	0.86	19	2.77	42	1.60	42
			9	1.19	21	1.11	24	0.92	21	0.90	23	0.90	22	0.88	23	2.04	27	1.35	32
			10	1.13	20	1.10	24	0.96	21	0.98	22	0.93	21	0.95	23	2.56	23	1.74	20

The axial force and bending moment ratios are now discussed. The A_3 and M_3 parameters are considered for that purpose. These parameters have the same meaning that D_3 and V_3 except that now axial forces and bending moments are considered instead of displacements and shears. As for the other parameters, figures for the two models, directions and deformation levels were developed for A_3 and M_3 but are not shown. Only the results in terms of fundamental statistics are shown for all cases (Table 9). The results, in general, indicate that the A_3 mean values are larger for elastic than for inelastic deformations and higher for the 3- than for the 10-level model; values close, or even larger than 2, are observed (interior columns). The uncertainty in the estimation is considerable and significantly larger for elastic behavior and the 3-level building, values of δ of up to 60% can be observed. Comparing the results of the axial force ratios here found (A_3) with those of the equivalence in strength (A_1) it is observed that the mean and coefficient of variation values are greater for the case of equivalence in weight. For the M_3 ratio, their mean values are smaller than unity, which in turn are greater for interior columns of the 10-level model. The coefficient of variation values range from 23 to 39%, showing an important uncertainty in the estimation. Significant differences are not observed between the statistics of M_3 for normal and principal components or for elastic and inelastic behavior. The mean values of bending moment ratios are smaller for the equivalence in terms of weight (M_3) than for the equivalence in terms of strength (M_1). However, the uncertainty in their estimation is larger for the equivalence in terms of weight.

Table 9. Statistics for the A_3 and M_3 parameters

MODEL		MEMBER LOCATION	A_3								M_3							
			NORMAL COMPONENT				PRINCIPAL COMPONENT				NORMAL COMPONENT				PRINCIPAL COMPONENT			
			X DIRECTION		Y DIRECTION		X DIRECTION		Y DIRECTION		X DIRECTION		Y DIRECTION		X DIRECTION		Y DIRECTION	
			μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC	INTERIOR	1.92	57	1.94	56	2.02	60	1.93	61	0.62	37	0.66	25	0.60	39	0.69	23
		EXTERIOR	1.14	39	1.20	39	1.08	45	1.19	36	0.57	38	0.60	24	0.55	38	0.63	23
	INELASTIC	INTERIOR	1.12	21	1.10	29	1.14	25	1.11	29	0.62	38	0.67	27	0.58	38	0.68	26
		EXTERIOR	0.95	28	1.04	24	0.93	34	1.02	24	0.56	37	0.60	25	0.53	37	0.63	28
2	ELASTIC	INTERIOR	1.50	47	1.40	45	1.50	47	1.44	46	0.78	27	0.70	29	0.78	28	0.71	26
		EXTERIOR	0.97	29	0.95	36	0.97	30	0.96	33	0.72	27	0.64	29	0.72	28	0.65	26
	INELASTIC	INTERIOR	0.99	21	0.97	10	0.94	18	0.98	9	0.77	24	0.73	31	0.79	27	0.75	33
		EXTERIOR	0.93	15	0.87	17	0.95	21	0.85	13	0.71	24	0.66	32	0.72	27	0.68	32

10. 2D MODELS WITH EQUIVALENT COLUMNS IN TERMS OF WEIGHT

The responses of buildings with W14 columns are compared with the corresponding responses obtained by considering W27 columns, when such buildings are modeled as plane frames and the equivalence between both column sizes is expressed in terms of weight. The parameters $D_4 = D_{2,M,W}/D_{2,L,W}$ and $V_4 = V_{2,M,W}/V_{2,L,W}$, are used to compare the interstory displacements and shears, respectively. The terms in the D_4 and V_4 expressions have the same meaning as that of D_2 and V_2, except that, as stated earlier, the equivalence of columns is expressed in terms of weight. Individual graphics for all cases were developed but only their statistics are presented (Table 10). Even though plots are not given, it can be said that, in general, the interstory displacements are larger when W14 columns are used, values larger than 3 are obtained for D_4 in many cases. For the case of the statistics, mean values of up to 1.84 are observed for D_4 from Table 10. They are greater for the stories close to the ground floor level. The V_4 mean values indicate that the interstory shears are larger when W27 columns are used. The uncertainty in the estimation is larger for D_4 than for V_4 and larger for the 3- than for 10-level model. No significant differences are observed between the statistics of elastic and inelastic behavior or normal and principal components. Results also indicate that the displacements ratios for the 2D modeling may be significantly greater for the equivalence in terms of weight when compared to those of equivalence in terms of strength. For shears however, the ratios are slightly larger for the equivalence in strength.

With the purpose of comparing axial forces and bending moments for buildings with both column sizes, the parameters A_4 and M_4 are used. Their statistics are presented in Table 11. The results indicate that the A_4 mean values for inelastic behavior are close to the unity while those of elastic behavior are generally greater than unity. The M_4 mean values are smaller than unity in all cases. As for the A_4 parameter, the mean values of the M_4 ratio resulted greater for the elastic case. The δ values of A_4 and M_4 show that there is a considerable dispersion, particularly for the elastic case. This is greater for the 3-level model. Comparing the A_4 mean values here found with those of the equivalence in terms of strength (A_2) it is observed, in general, that they are larger for the equivalence in terms of weight. The mean values of moment ratios are larger for the equivalence in strength. The uncertainty in the estimation of both parameters resulted greater for the case of equivalence in terms of weight.

Table 10. Statistics for the D_4 and V_4 parameters

MODEL		STORY	D_4				V_4			
			NORMAL COMPONENT		PRINCIPAL COMPONENT		NORMAL COMPONENT		PRINCIPAL COMPONENT	
			μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC	1	1.64	44	1.69	43	0.89	42	0.90	41
		2	1.24	39	1.28	41	0.86	39	0.89	42
		3	1.13	38	1.15	41	0.87	36	0.87	38
	INELASTIC	1	1.77	38	1.84	40	0.85	16	0.83	21
		2	1.32	37	1.39	42	0.86	13	0.86	22
		3	1.18	42	1.23	45	0.85	18	0.84	23
2	ELASTIC	2	1.41	26	1.47	28	0.89	26	0.92	28
		3	1.15	29	1.18	30	0.89	29	0.91	30

		4	1.13	25	1.14	25	0.88	25	0.88	25
		5	1.18	22	1.16	21	0.91	23	0.90	20
		6	1.27	22	1.28	23	0.95	22	0.96	22
		7	1.18	22	1.21	22	0.91	22	0.94	22
		8	1.18	24	1.22	26	0.87	24	0.90	25
		9	1.19	23	1.19	24	0.90	23	0.90	24
		10	1.11	22	1.11	20	0.91	22	0.90	20
	INELASTIC	2	1.40	21	1.48	24	0.88	16	0.91	17
		3	1.16	29	1.24	31	0.88	18	0.91	19
		4	1.12	26	1.16	27	0.88	17	0.89	16
		5	1.16	23	1.18	23	0.92	19	0.91	16
		6	1.22	24	1.22	26	0.95	19	0.94	20
		7	1.10	20	1.11	23	0.92	15	0.91	17
		8	1.12	24	1.15	27	0.87	18	0.88	20
		9	1.16	23	1.18	24	0.91	18	0.90	18
		10	1.08	21	1.10	19	0.90	18	0.90	17

From the results presented in this section of the papers and in the others, it can be summarized that the ratio of interstory displacements of steel buildings with PMRF to the corresponding displacements of the buildings with W27 columns are always larger than unity, values close to 2 are observed in some particular cases (Fig. 7a). The implication of this is that the use of deep columns may help to significantly reduce the interstory drifts, which is an important parameter considered in building seismic design codes. As stated earlier, this is particularly important for tall steel buildings where the design is controlled by this limit state. The corresponding ratios in terms of average interstory shears and interstory shears for exterior individual frames are slightly smaller than unity implying a similar shear for the building with both column sizes, but for interior frames the shears are always significantly larger for the buildings with W14 columns. The bending moments are always larger for the buildings with W27 columns, however, the axial forces are significantly larger for the building with W14 columns for some cases.

The seismic responses of the 3D buildings with deep columns are also compared with those of the 2D buildings with deep columns. However, the results are not presented, only the main conclusions are commented. It is found that the interstory displacements and shears may be significantly greater for the plane models than for the three-dimensional models. For the case of axial forces, they resulted also much larger for plane models practically in all cases. Bending moments resulted smaller for plane models in some cases. The above comments are valid for both criteria of equivalence in columns. The implication of this is that the modeling a 3D building as a plane frame for purposes of seismic analysis may result in a very conservative design

Table 11. Statistics for the A_4 and M_4 parameters

MODEL		MEMBER LOCATION	A_4				M_4			
			NORMAL COMPONENT		PRINCIPAL COMPONENT		NORMAL COMPONENT		PRINCIPAL COMPONENT	
			μ	δ	μ	δ	μ	δ	μ	δ
1	ELASTIC	INTERIOR	1.26	52	1.19	43	0.71	45	0.72	42
		EXTERIOR	1.18	52	1.21	55	0.66	45	0.67	42

	INELASTIC	INTERIOR	0.96	15	1.02	27	0.66	19	0.67	24
		EXTERIOR	0.99	16	1.02	24	0.62	21	0.63	25
2	ELASTIC	INTERIOR	1.14	32	1.18	34	0.87	27	0.90	29
		EXTERIOR	1.00	17	1.02	20	0.80	27	0.83	29
	INELASTIC	INTERIOR	0.98	7	0.99	6	0.85	21	0.89	24
		EXTERIOR	0.96	10	0.97	11	0.79	19	0.82	21

12. CONCLUSIONS

The ratio of the seismic responses of steel buildings with perimeter moment resisting frames (PMRF) with medium columns (W14) to the corresponding responses of steel buildings with large columns (W27) is studied. Two models of steel buildings considered in the steel SAC project are used for this purpose. The W27 columns are selected according to two criteria: (1) equivalent resistance (the W14 column weight is approximately 60% higher and therefore represents a higher cost), and (2) equivalent weight (equal cost). It is shown that the W27 columns have not problems of lateral torsional buckling, local buckling or shear buckling in panel zone. The comparison is made in terms of global (interstory displacements and shears), and local (axial loads and bending moments) response parameters. Plane and three-dimensional models are considered. The structural models are subjected to the action of twenty seismic records of earthquakes that have been previously selected taking into account their intensity, frequency contents and duration of the strong phase.

The results indicate that the two-dimensional modeling of three-dimensional structures may result in very conservative designs. It is also observed that the responses may be greater for W14 or W27 columns case, depending on the deformation level, response parameter and the structural modeling under consideration. The ratio of interstory displacements of steel buildings with W14 columns to the corresponding displacements of the buildings with W27 columns are always larger than unity, values larger than 2 are observed in some particular cases. The implication of this is that the use of deep columns may help to significantly reduce the interstory drifts, which is an important parameter considered in building seismic design codes. This is particularly important for tall steel buildings where the design is controlled by this limit state.

The corresponding ratios in terms of average interstory shears, and interstory shears for exterior individual frames, are slightly smaller than unity implying a similar shear for the building with both column sizes. For interior frames, however, they are always significantly larger than unity for the buildings with W14 columns, values close to 3 are observed in some cases. The implication of this is that W27 columns attract higher interstory shears than W14 columns, accordingly the interstory shears at the interior gravity frames (GF) of models with W27 columns are much smaller than those of the models with W14 columns. It helps to counteract the no conservative effect that results in the design practice of the structural system under consideration when lateral seismic forces are not considered in the interior frames. The bending moments are always larger for the buildings with W27 columns, however, the axial forces are significantly larger for the building with W14 columns for some cases. Significant differences are not observed between normal and principal components or between elastic and inelastic behavior, for the case of global response parameters and for the case of moments in columns. It is concluded that, the performance of steel buildings with PMRF with W27 columns may be superior to that of buildings with PMRF with W14 columns, using therefore less weight and representing a lower cost.

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