*ISET Journal of Earthquake Technology, **Technical Note**, Vol. 46, No. 3-4, Sept.-Dec. 2009, pp. 133–146*

SOME ISSUES REGARDING THE STRUCTURAL IDEALIZATION OF PERIMETER MOMENT-RESISTING STEEL FRAMES

A. Reyes-Salazar*, E. Bojórquez*, A. López-Barraza*, D. De Leon-Escobedo** and A. Haldar***

*Facultad de Ingeniería, Universidad Autónoma de Sinaloa
Calzada de las Américas y Boulevard Universitarios S/N, Ciudad Universitaria, Culiacán, CP 80040, México
**Facultad de Ingeniería, Universidad Autónoma del Estado de México
Cerro de Coatepec S/N, Ciudad Universitaria, Toluca, CP 50000, México
***Department of Civil Engineering and Engineering Mechanics
The University of Arizona, Tucson, AZ 85721, U.S.A.

ABSTRACT

To improve economy and increase efficiency in designing the steel buildings made with W-shape members, the use of two-dimensional moment-resisting steel frames (MRSFs) in the perimeter, to carry seismic loads, and of interior gravity frames (IGFs), to carry gravity loads, has become very popular. This provides a simpler frame to analyze and design. The behavior of such buildings is addressed in this paper. Two model buildings suggested by FEMA are used in the study. Linear and nonlinear global and local responses are calculated. The results indicate that modelling 3-D structures with 2-D perimeter frames is conservative and that ignoring the lateral load carrying capacity of IGFs or ignoring the rigidity of connections in those adds several levels of conservatism in the overall design. This study also indicates that IGFs may need to carry up to 35% of the total lateral load caused by the seismic excitation. An implication of this is that the members in IGFs may not be able to carry this unexpected load.

KEYWORDS: Semi-Rigid Connections, Pinned Connections, Steel Buildings, Perimeter Moment Frames, Interior Gravity Frames

INTRODUCTION

To improve economy and increase efficiency in designing the steel buildings, made with W-shape members with very different section properties about the major and minor axes, to carry lateral seismic loads, the use of perimeter moment-resisting steel frames (MRSFs) has become very popular. As will be discussed in more detail later, following the Northridge earthquake of 1994, Federal Emergency Management Agency (FEMA) (FEMA, 2000) suggested structural arrangements and member sizes of three such model buildings for promoting their use. Those can be considered as the benchmark models. The underlying concept of such structural arrangements is that MRSFs are provided at the perimeter of the building to carry the seismic load, and interior gravity frames (IGFs) are provided to carry only the gravity loads. Because of their high ductility capacity, the MRSFs provide a very efficient lateral load resisting system in two orthogonal directions. Individually, each frame in the three-dimensional structures can be considered to behave in two dimensions, thus providing a simpler frame to analyze and design. Moment-resisting connections (MRCs) are very expensive to build, at least in the U.S. They are also not suitable for the weak-axis connections (FEMA, 2000). The use of MRCs is only limited to the MRSFs in the buildings with perimeter MRSFs; connections in IGFs are generally not considered to be of moment-resisting type. Thus, the use of fewer MRSFs with MRCs in the perimeter and IGFs without any MRC in the interior introduces an overall economy in the design. Two typical structural arrangements used by FEMA (2000) are shown in Figure 1.

FEMA did not explicitly suggest how to design the buildings with perimeter MRSFs. The behaviour of such buildings, which are excited by seismic loads, is an open question and needs critical evaluation by the profession. The main purpose of this paper is to provide such an evaluation and establish their desirable behavior.

There are several issues that deserve the attention of the profession. The lateral load carrying capacity of IGFs is generally ignored in the buildings with perimeter MRSFs. For the static application of the seismic loads, this assumption can be considered conservative. Due to the action of rigid floor diaphragms, the IGFs are expected to undergo the same lateral deformations as the MRSFs.

Consequently, the contribution of the columns in IGFs to the lateral resistance could be significant, particularly for the buildings with relatively fewer MRCs. However, can this conservatism of ignoring the lateral load carrying capacity of IGFs still be valid for the dynamic application of the seismic loads?

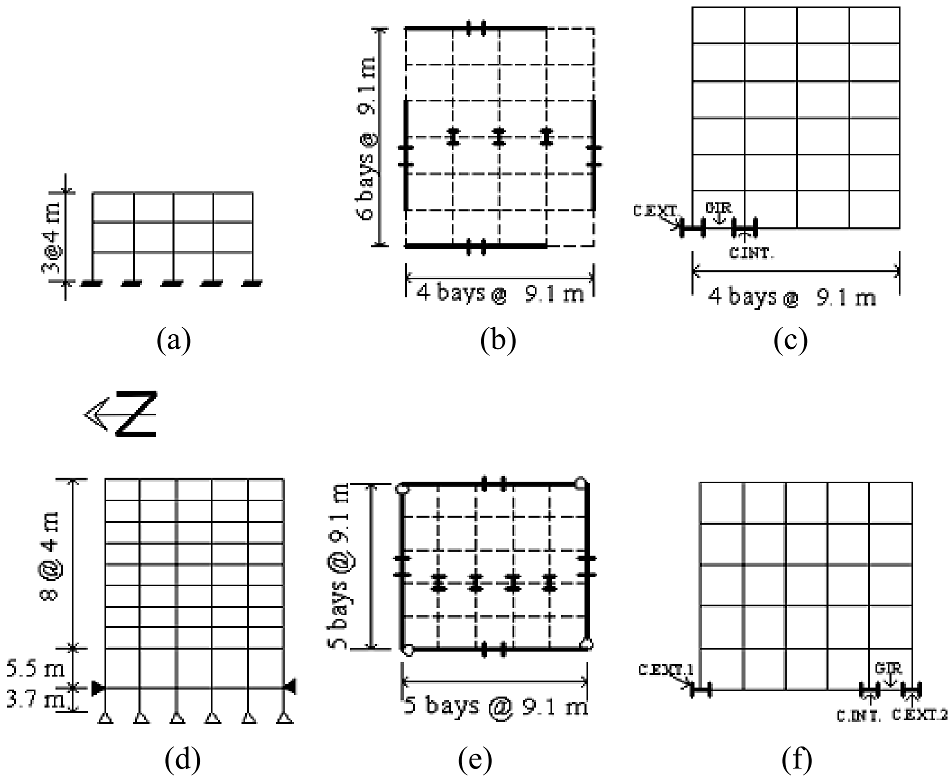


Fig. 1 Elevation, plan and element location for Models 1 and 2: (a) elevation of Model 1, (b) plan of Model 1, (c) studied elements of Model 1, (d) elevation of Model 2, (e) plan of Model 2, (f) studied elements of Model 2

Modeling MRSFs in the perimeter of a building as plane frames may not represent the actual behavior of the structure since the participation of some elements is not considered. The dynamic properties in terms of stiffness, natural frequencies, damping or energy dissipation characteristics, etc. for two-dimensional (2-D) and three-dimensional (3-D) modeling of such structures are expected to be different. The corresponding structural responses are also expected to be different. However, these differences are unknown at this time and need to be quantified.

Another issue generally overlooked by the profession is the rigidity of connections in steel frames and its influence in the estimation of dynamic response behavior. Connections in MRSFs are considered to be fully restrained (FR) and in IGFs to be perfectly pinned (PP). Despite these classifications, almost all steel connections used in real frames are essentially semi-rigid (SR) with different rigidities. It has been established in the profession, both theoretically and experimentally, that these connections exhibit semi-rigid nonlinear behavior even if the applied static and dynamics loads are very small (Reyes-Salazar and Haldar, 2000). The FR and PP connection consideration is nothing but an assumption made to simplify the calculations and is a major weakness in the current analytical procedures. These simplifications may result in an erroneous global and local response behavior because in reality FR connections possesses some flexibility and PP connections possesses some rigidity. There is some evidence that shear connections can transmit up to 30% of the plastic moment capacity (FEMA, 2000) of the beams they connect. The contribution of these connections to the structural response can be of much importance if composite action of the slab is considered (Reyes-Salazar and Haldar, 1999; Liu and Astaneh-Asl, 2000).

Ideally, to study the response behavior of steel buildings with perimeter MRSFs and interior IGFs, structural systems need to be represented as realistically as possible, preferably in 3-D, and then responses need to be estimated by exciting them with measured seismic time-histories. The implications of several modeling assumptions discussed earlier can then be established by comparing the results with the benchmark results for the ideal case. This study will also provide some design guidelines for the profession to consider. Some of the issues specifically addressed in this paper are (a) the accuracy of modeling three-dimensional buildings as two-dimensional perimeter plane frames for seismic design, (b) the level of contribution of IGFs to the lateral resistance of the overall building, and (c) the effect of the stiffness of the connections of IGFs on structural responses. To document the results numerically, global responses in terms of the base shear and interstory displacements and local responses in terms of the resultant stresses at individual members are presented here.

MATHEMATICAL FORMULATION

To satisfy the objectives of this study, the nonlinear seismic responses of the 2-D and 3-D steel building models are evaluated as accurately as possible by using a sophisticated and efficient assumed stress-based finite element algorithm developed by the authors and their team at the University of Arizona. The concept used is discussed in detail elsewhere (Gao and Haldar, 1995; Reyes-Salazar, 1997). This procedure estimates nonlinear seismic responses in time domain considering all major sources of nonlinearities, including the geometric, material, and appropriate rigidities of connection and support conditions. In this approach, an explicit form of the tangent stiffness matrix is derived without any numerical integration. Further, fewer elements can be used in describing a large-deformation configuration without sacrificing any accuracy, and the material nonlinearity and the nonlinearity introduced by SR connections can be incorporated without losing its basic simplicity. It gives very accurate results for framed structures and is very efficient compared to the commonly used displacement-based approaches. The procedure and its algorithm have been extensively verified using the available theoretical and experimental results (Reyes-Salazar and Haldar, 2001a, 2001b).

CONNECTION FLEXIBILITY

Connections can be treated as the structural elements that transmit resultant stresses (axial and shear forces, torsion and bending moments) between beams and columns. The rigidity of connections is generally represented by the relationship between the bending moment at the connections and the corresponding relative rotation, referred to as the moment-relative rotation or M-θ curves. Many mathematical forms used to define the M-θ curves are available in the literature, including the piecewise linear, polynomial, exponential, B-spline forms and the Richard model (Richard, 1993; Reyes-Salazar, 1997; Reyes-Salazar and Haldar, 1999). The four-parameter Richard model, as shown in Figure 2, appears to be most appropriate for our evaluation. It was developed using the actual worldwide test data. When a connection is defined in terms of member sizes and bolts and/or welds, a commercially available computer program, known as PRCONN, may be used to generate the appropriate M-θ curve corresponding to the Richard model (Richard, 1993). The PRCONN program was used in this study to develop the required M-θ curve. According to the Richard model, the M-θ curve is given by

$$M = \frac{\left(k-k_p\right)\theta}{\left(1+\left|\frac{\left(k-k_p\right)\theta}{M_0}\right|^N\right)^{\frac{1}{N}}} + k_p\theta \tag{1}$$

where k is the initial or elastic stiffness, k_p is the plastic stiffness, M_0 is the reference moment, and N is the curve shape parameter. The physical definitions of these parameters are shown in Figure 2.

Equation (1) represents the M-θ curve when the load is increasing monotonically. When a structure is excited by the dynamic or seismic loading, some of the connections are expected to be loading and others are expected to be unloading and reloading. Experimental and theoretical studies related to the unloading and reloading behavior of the M-θ curve, as shown in Figure 3, are rare. This subject has been addressed in the literature (Colson, 1991; El-Salti, 1992). For the present study, the unloading and reloading behavior of the M-θ curve is essential. As in the past (Reyes-Salazar, 1997; Reyes-Salazar and Haldar, 1999, 2000, 2001a, 2001b), in the present study, the monotonic loading behavior is represented by the Richard curve and the Masing rule is used to theoretically develop the unloading and reloading sections of the M-θ curve. A general class of Masing models can be defined with a virgin loading curve as

$$f\left(M,\theta\right)=0 \tag{2}$$

and its unloading and reloading curve can be described by the following equation:

$$f\left(\frac{M-M_{\alpha}}{2},\frac{\theta-\theta_{\alpha}}{2}\right)=0 \tag{3}$$

where (M_a,θ_a) denotes the load reversal point as shown in Figure 3.

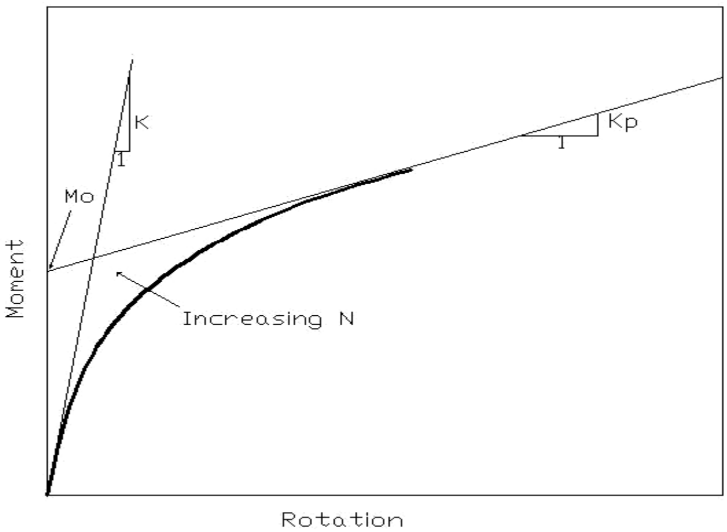


Fig. 2 Parameters of Richard Model

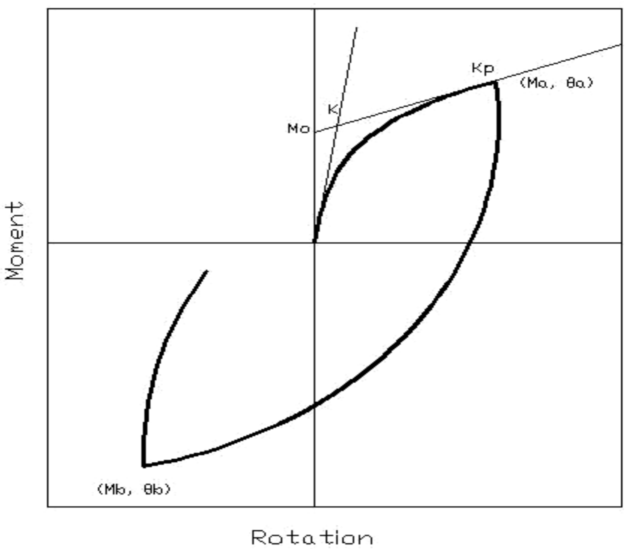


Fig. 3 Loading, unloading, and reloading at SR connections

Using the Masing rule and the Richard model represented by Equation (1), the mathematical representation for the unloading and reloading behavior of a connection can be expressed as

$$M=M_{a}-\frac{\left(K-K_{p}\right)\left(\theta_{a}-\theta\right)}{\left(1+\left|\frac{\left(K-K_{p}\right)\left(\theta_{a}-\theta\right)}{2M_{0}}\right|^{N}\right)^{\frac{1}{N}}}-K_{p}\left(\theta_{a}-\theta\right) \tag{4}$$

If (M_b, θ_b) is the next reversal point, as shown in Figure 3, the reloading relation between M and θ can be obtained by simply replacing (M_a, θ_a) with (M_b, θ_b) in Equation (4). Thus, Equation (1) is used if the connection is loading; if it is unloading or reloading, Equation (4) should be used instead.

In the computer program, as discussed earlier and developed by the authors and their team, a new element is introduced to represent a SR connection. If the loading, unloading and reloading behavior of each connection is defined properly, the program computes the required responses accordingly. For a dynamic loading, SR behavior also adds a source of energy dissipation (Reyes-Salazar and Haldar, 2000). The stage is now set to study the issues discussed earlier.

STRUCTURAL AND LOAD MODELS

1. Structural Models

As part of the SAC steel project, FEMA (2000) commissioned three consulting firms to perform the design of several building models. They considered 3-, 9- and 20-story buildings. These buildings are supposed to satisfy all code requirements that existed at the time of evaluation for the following three cities: Los Angeles and Seattle which satisfied the 1994 Uniform Building Code (ICBO, 1994) requirements, and Boston which satisfied the Building Officials & Code Administration (BOCA, 1993) requirements. The 3- and 9-story buildings located in the Los Angeles area are considered in this study for numerical evaluations in order to address all the issues discussed earlier. They will be denoted hereafter as Models 1 and 2, respectively. As mentioned earlier, these are the benchmark models to be considered by other researchers. They provide a unique opportunity to study the behavior of steel buildings with perimeter MRSFs and interior IGFs.

The fundamental periods of the buildings are estimated to be 1.03 and 2.34 s, respectively. The elevations of the models are shown in Figures 1(a) and 1(d), and their plans are shown in Figures 1(b) and 1(e), respectively. In these figures, the MRSFs are represented by the continuous lines and the IGFs are represented by the dashed lines. For Model 2, the perimeter frames meet at a corner. In this case, the beam-column connections are considered to be pinned to eliminate weak-axis bending (see Figure 1(e)). As shown in Figure 1, the buildings are essentially symmetrical in plan and no significant torsional moments are expected to occur. Resultant stresses are estimated for both columns and girders of the MRSFs. The particular elements considered in the study are located at the ground floor level and are shown in Figures 1(c) and 1(f) for Models 1 and 2, respectively. The interior and exterior columns as well as girders of the MRSFs are indicated in these figures. The sizes of beams and columns, as reported by FEMA for the two models, are given in Table 1. The columns of the MRSFs are considered to be fixed at the base for Model 1 and pinned for Model 2, as considered in the FEMA report. In all of these frames, the columns are made of Grade-50 steel and the girders are of A36 steel. For both models, the columns in IGFs are considered to be pinned at the base. All columns in the perimeter MRSFs bend about the strong axis, and the strong axis of the gravity columns is oriented in the N-S direction, as indicated in Figures 1(b) and 1(e). The designs of the MRSFs in the two orthogonal directions were practically same. Additional information for the two models can be obtained from the FEMA report.

The buildings are modeled as multi-degree-of-freedom systems (MDOFs). Each column is represented by one element and each girder of the perimeter MRSFs is represented by two elements, thus having a node at the mid-span. Each node is considered to have six degrees of freedom when the buildings are modeled in three dimensions. Both PP and SR connections are considered for the IGFs. As mentioned earlier, an additional element is needed to represent each SR connection. These connections are considered only for the bending with respect to the strong axis of the gravity columns. Therefore, they are oriented in the E-W direction.

2. Earthquake Loading

Dynamic responses of a structure excited by different earthquake time-histories, even when the time-histories are normalized with respect to the respective peak ground accelerations, are expected to be different due to their different frequency contents. Thus, evaluating structural response for an earthquake ground motion may not reflect the structural behavior properly. To study the responses of the two models comprehensively, they are excited in time domain by 20 recorded earthquake motions recorded at different locations. The characteristics of these earthquake time-histories are given in Table 2. As shown in the table, the predominant periods of the earthquake ground motions vary from 0.11 to 1.0 s. The predominant period for each motion is defined as the period where the largest peak in the elastic response spectrum occurs. The earthquake time-histories are obtained from the datasets of the National Strong Motion Program (NSMP) of the United States Geological Survey (USGS). Additional information on these earthquake ground motions can be obtained from the USGS. Damping is considered to be 5% of the critical damping, as this is the value used in developing the code provisions in the U.S. Since the structures and loadings are completely defined now, the stage is set to study the three issues identified earlier.

Table 1: Beam and Column Sections for Models 1 and 2

Model	Moment-Resisting Frames				Gravity Frames		
	Story	Columns		Girders	Columns		Beams
		Exterior	Interior		Below Penthouse	Others	
1	1/2	W14×257	W14×311	W33×118	W33×118	W14×68	W18×35
	2/3	W14×257	W14×312	W30×116	W30×116	W14×68	W18×35
	3/Roof	W14×257	W14×313	W24×68	W24×68	W14×68	W16×26
2	−1/1	W14×370	W14×500	W36×160	W36×160	W14×193	W18×44
	1/2	W14×370	W14×500	W36×160	W36×160	W14×193	W18×35
	2/3	W14×370	W14×500, W14×455	W36×160	W36×160	W14×193, W14×145	W18×35
	3/4	W14×370	W14×455	W36×135	W36×135	W14×145	W18×35
	4/5	W14×370, W14×283	W14×455, W14×370	W36×135	W36×135	W14×145, W14×109	W18×35
	5/6	W14×283	W14×370	W36×135	W36×135	W14×109	W18×35
	6/7	W14×283, W14×257	W14×370, W14×283	W36×135	W36×135	W14×109, W14×82	W18×35
	7/8	W14×257	W14×283	W30×99	W30×99	W14×82	W18×35
	8/9	W14×257, W14×233	W14×283, W14×257	W27×84	W27×84	W14×82, W14×48	W18×35
	9/Roof	W14×233	W14×257	W24×68	W24×68	W14×48	W16×26

Table 2: Earthquake Models

No.	Place	Year	Station	*T* (s)	Epicentral Dist. (km)	*M*	PGA (mm/s^2)
1	1317 Michoacan, México	1985	Paraíso	0.11	300	8.1	800
2	1634 Mammoth Lakes, USA	1980	Mammoth H.S. Gym	0.12	19	6.5	2000
3	1634 Mammoth Lakes, USA	1980	Convict Creek	0.19	18	6.5	3000
4	1317 Michoacan, México	1985	Infiernillo N-120	0.21	67	8.1	3000
5	1317 Michoacan, México	1985	La Unión	0.32	121	8.1	1656
6	1733 El Salvador	2001	Relaciones Ext.	0.34	96	7.8	2500
7	1733 El Salvador	2001	Relaciones Ext.	0.41	95	7.8	1500
8	1634 Mammoth Lakes	1980	Long Valley Dam	0.42	13	6.5	2000
9	2212 Delani Fault, AK, USA	2000	K2-02	0.45	281	7.9	115
10	0836 Yountville, CA, USA	2000	Redwood City	0.46	95	5.2	90
11	0408 Dillon, MT, USA	2005	Kalispell	0.51	338	5.6	51
12	1317 Michoacan, Mexico	1985	Villita	0.53	80	8.1	1225
13	1232 Northridge, USA	1994	Hall Valley	0.54	25	6.4	2500
14	2115 Morgan Hill, USA	1984	Hall Valley	0.61	14	6.2	2000
15	2212 Delani Fault, AK, USA	2002	K2-04	0.62	290	7.9	133
16	0836 Yountville, CA, USA	2000	Dauville F.S. CA	0.63	73	5.2	144
17	0836 Yountville, CA, USA	2000	Pleasant Hill F.S. 1	0.71	92	5.2	74
18	0836 Yountville, CA, USA	2000	Pleasant Hill F.S. 2	0.75	58	5.2	201
19	2212 Delani Fault, AK, USA	2002	Valdez City Hall	0.85	272	7.9	260
20	1715 Parkfield, CA, USA	2004	Hollister City Hall	1.01	147	6.0	145

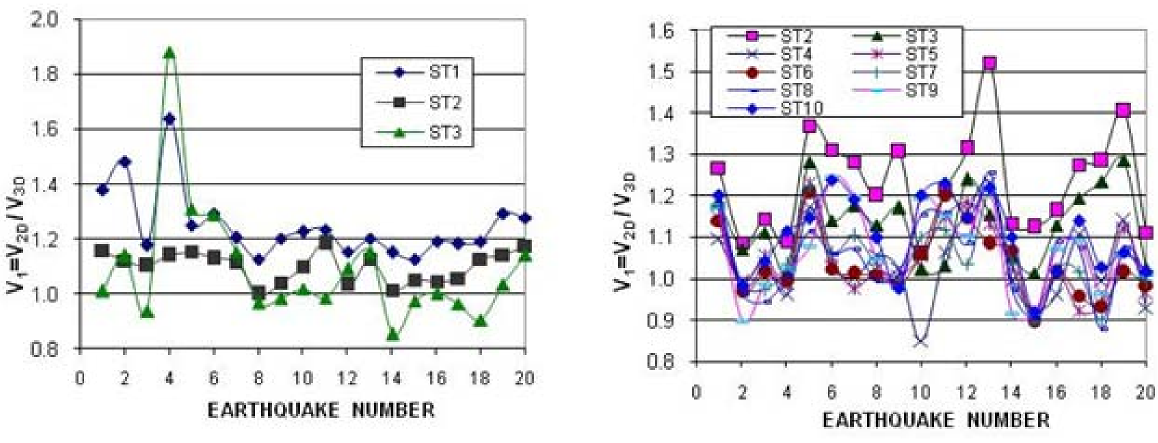
ISSUE 1—MODELING THE 3-D BUILDINGS AS 2-D PLANE FRAMES

The global and local seismic responses of idealized 2-D perimeter frames are compared with those of the more realistic 3-D frames for Models 1 and 2 in this section. The global responses are studied in terms of the interstory shear and interstory displacement and the local responses are evaluated in terms of the resultant stresses in some members at the base of the structure.

The interstory shear is considered first. The shear ratio V_1, defined as V_{2D} / V_{3D}, is introduced for this purpose. V_{2D} represents the maximum shear resisted by all the columns in a story in a given direction when the building is modeled as a plane frame, and V_{3D} represents the same response but for the building modeled as a 3-D structure. Since each earthquake has two horizontal components, the V_1 ratio is estimated for both horizontal directions. The horizontal component containing the peak acceleration is applied in the N-S direction and the other horizontal component is applied in its perpendicular direction, i.e., in the E-W direction.

The V_1 ratio is plotted for all 20 earthquake ground motions in Figures 4(a) and 4(b) for Models 1 and 2, respectively, when excited in the N-S direction, for all three stories of Model 1 and all nine stories of Model 2. In these figures, the symbol ST is used to represent the word "story". It is observed that the V_1 values vary significantly from one model to another and from one story to another without showing any trend. In most of the cases, V_1 values are larger than unity, sometimes as large as 1.4, thus indicating that the interstory shears are larger for the 2-D model than for the 3-D model. This is due to the strength and stiffness contributions of some elements that are considered in the 3-D model but not in the 2-D model. This also points out that the dynamic characteristics of 2-D and 3-D representations are different and this aspect should not be overlooked. It is well known that the response of 3-D buildings, when subjected to strong motions at the base, depends on many factors, specifically on the spatial distribution of strength, stiffness and mass, the frequency content of the excitation, the energy dissipation characteristics (or damping) in the linear and nonlinear responses, etc. It is important to emphasize that a building, when modeled as a 3-D frame, is expected to have different natural frequencies than when it is modeled as a 2-D frame and will thus respond differently, when subjected to the same excitation. This is the reason why 20 different recorded time-histories and different structural arrangements are considered in this study.

In general, V_1 values are larger for the story at the ground level. The statistics, in terms of mean, standard deviation, and coefficient of variation (COV) of V_1, are summarized in Table 3. In most cases, COVs are relatively small; those are less than 0.10. Plots similar to Figure 4 were developed for the excitation in the E-W direction but are not shown here. The major observations made for the excitation in the N-S direction are also valid for the excitation in this direction.



(a) (b)

Fig. 4 V_1 values for (a) Model 1 and (b) Model 2

The frames for both models did not develop any plastic hinge when excited by any of the 20 recorded motions, thus indicating a high level of conservatism used in the design of these model buildings. To study the effect of inelastic behavior on the V_1 ratio, the actual time-histories were scaled up to produce some yielding in both models. Based on the past experience and for the uniformity of comparison, all the actual time-histories were scaled up by the trial and error procedure to develop a maximum average interstory drift of about 1%, instead of tracking the total number of plastic hinges developed. It was observed that about 1–5 plastic hinges were formed in the models when they developed the desired drift. Results were plotted similar to Figure 4 for both models and for both directions, but those are not shown here. It is observed that, since yielding was not very significant, the V_1 values remain practically same for the elastic and inelastic behavior.

Table 3: Statistics for Ratios V_1 and D_1

Model		Story	Statistics of V_1						Statistics of D_1					
			N-S Direction			E-W Direction			N-S Direction			E-W Direction		
			μ	σ	COV	μ	σ	COV	μ	σ	COV	μ	σ	COV
1	Elastic	1	1.23	0.12	0.10	1.05	0.06	0.06	1.20	0.09	0.08	1.12	0.07	0.06
		2	1.10	0.05	0.05	1.04	0.07	0.07	1.10	0.04	0.04	1.01	0.06	0.06
		3	1.10	0.22	0.20	1.00	0.08	0.08	1.10	0.15	0.14	0.98	0.07	0.07
	Inelastic	1	1.22	0.12	0.10	1.13	0.08	0.07	1.20	0.09	0.08	1.11	0.09	0.08
		2	1.09	0.05	0.05	1.09	0.05	0.05	1.10	0.04	0.04	1.06	0.11	0.10
		3	1.08	0.19	0.18	1.03	0.08	0.08	1.09	0.14	0.13	1.02	0.12	0.12
2	Elastic	2	1.18	0.08	0.07	1.36	0.07	0.05	1.16	0.08	0.07	1.16	0.06	0.05
		3	0.97	0.07	0.07	1.04	0.04	0.04	1.01	0.07	0.07	1.1	0.05	0.05
		4	1.00	0.06	0.06	1.05	0.05	0.05	0.99	0.06	0.06	1.00	0.05	0.05
		5	1.02	0.08	0.08	1.05	0.04	0.04	1.02	0.08	0.08	1.03	0.05	0.05
		6	1.03	0.07	0.07	1.07	0.04	0.04	1.02	0.07	0.07	1.00	0.04	0.04
		7	0.99	0.08	0.08	1.02	0.04	0.04	1.00	0.08	0.08	1.01	0.05	0.05
		8	1.02	0.08	0.08	1.07	0.05	0.05	1.01	0.08	0.08	1.02	0.05	0.05
		9	1.03	0.09	0.09	1.09	0.06	0.06	1.02	0.09	0.09	1.03	0.06	0.06
		10	1.03	0.09	0.09	1.03	0.06	0.06	1.03	0.08	0.08	1.08	0.06	0.06
	Inelastic	2	1.17	0.08	0.07	1.36	0.07	0.05	1.16	0.08	0.07	1.16	0.06	0.05
		3	0.97	0.06	0.06	1.04	0.04	0.04	1.01	0.07	0.07	1.11	0.06	0.05
		4	0.99	0.06	0.06	1.05	0.05	0.05	0.99	0.06	0.06	1.01	0.06	0.06
		5	1.02	0.07	0.07	1.05	0.04	0.04	1.02	0.08	0.08	1.03	0.05	0.05
		6	1.03	0.07	0.07	1.06	0.04	0.04	1.02	0.07	0.07	1.00	0.04	0.04
		7	0.99	0.08	0.08	1.02	0.04	0.04	1.00	0.08	0.08	1.01	0.04	0.04
		8	1.01	0.07	0.07	1.07	0.05	0.05	1.00	0.08	0.08	1.02	0.05	0.05
		9	1.02	0.08	0.08	1.09	0.06	0.06	1.02	0.09	0.09	1.03	0.06	0.06
		10	1.02	0.08	0.08	1.03	0.06	0.06	1.03	0.08	0.08	1.08	0.06	0.06

Similar to the interstory shear V_1, a new parameter D_1, defined as the ratio of the interstory displacements of the 2-D and 3-D models, is introduced next. The statistics of D_1 for excitations in the two directions are presented in Table 3. It may be observed that the conclusions made earlier for V_1 are also applicable to D_1, i.e., the mean values of D_1 are larger than unity in many cases and that those are quite similar for the elastic and inelastic behavior. The level of uncertainty for the elastic as well as inelastic behavior is similar for both parameters. As discussed earlier, to study the 2-D and 3-D modeling effects at the local element level, similar to V_1 and D_1, the ratios of axial loads (i.e., A_1) and moments (i.e., M_1) at some columns at the base are also estimated. Their statistics are presented in Table 4. These results indicate that the mean values of A_1 and M_1 are always greater than 1 and could be as large as 2.34, thus indicating that 2-D modeling will introduce significantly more stresses in the members than the 3-D modeling. To satisfy the stress requirement, therefore, larger-size members may be necessary. Moreover, in general, the record-to-record variability in A_1 and M_1 is much larger than that in V_1 and D_1. A large COV, as high as 0.68, can be observed in some members at the local level. This is expected. Global responses have some "averaging effect" since they depend on the contributions of many members, thus reducing the overall level of uncertainty; local responses, however, are estimated at a point in a member, thus producing a higher level of uncertainty.

Table 4: Statistics for Ratios A_1 and M_1

Model		Member*	Statistics of A_1						Statistics of M_1					
			N-S Direction			E-W Direction			N-S Direction			E-W Direction		
			μ	σ	COV	μ	σ	COV	μ	σ	COV	μ	σ	COV
1	Elastic	Girder	1.14	0.20	0.18	1.14	0.07	0.06	1.16	0.18	0.16	1.14	0.06	0.05
		C.Ext	1.11	0.06	0.05	1.23	0.11	0.09	1.17	0.04	0.03	1.27	0.12	0.09
		C.Int	1.35	0.10	0.07	1.23	0.1	0.08	1.23	0.10	0.08	1.25	0.08	0.06
	Inelastic	Girder	1.14	0.21	0.18	1.10	0.06	0.05	1.15	0.22	0.19	1.12	0.07	0.06
		C.Ext	1.09	0.05	0.05	1.22	0.11	0.09	1.11	0.08	0.07	1.22	0.13	0.11
		C.Int	1.49	0.41	0.28	1.22	0.11	0.08	1.51	0.43	0.28	1.24	0.10	0.08
2	Elastic	Girder	2.00	0.14	0.07	1.06	0.07	0.07	2.03	0.14	0.07	1.11	0.05	0.05
		C.Ext.1	1.08	0.10	0.09	1.21	0.09	0.08	1.08	0.11	0.10	1.21	0.11	0.09
		C.Int	1.31	0.21	0.16	1.14	0.08	0.07	1.33	0.18	0.14	1.14	0.11	0.10
		C.Ext.2	1.04	0.08	0.08	1.18	0.08	0.07	1.04	0.08	0.08	1.23	0.09	0.07
	Inelastic	Girder	2.32	0.97	0.42	1.09	0.33	0.30	2.34	0.47	0.20	1.12	0.31	0.28
		C.Ext.1	1.32	0.62	0.47	1.70	1.16	0.68	1.32	0.62	0.47	1.65	0.56	0.34
		C.Int	2.14	0.68	0.32	1.36	0.57	0.42	2.17	0.54	0.25	1.41	0.57	0.40
		C.Ext.2	1.12	0.39	0.35	1.41	0.6	0.43	1.14	0.39	0.34	1.39	0.50	0.36

*See Figures 1(c) and 1(d) for the locations of members

In summary, the results clearly indicate that the global and local responses obtained for the buildings modeled as 2-D frames will overestimate the seismic response, or in other words, will produce more conservative designs.

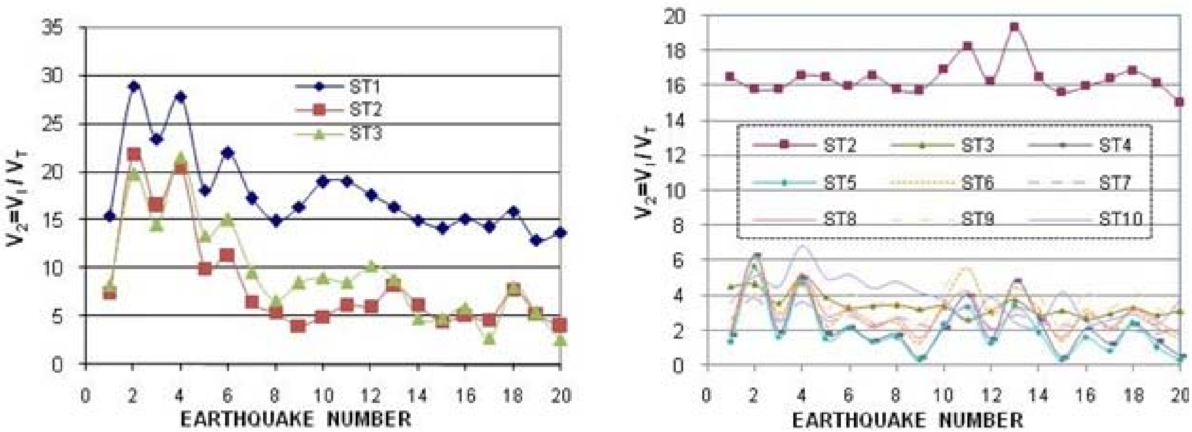
ISSUE 2—CONTRIBUTION OF GRAVITY FRAMES TO THE LATERAL RESISTANCE

1. Gravity Frames with PP Connection

The level of contribution of IGFs to the lateral resistance of the 3-D model is studied in this section of the paper. The connections of the beam and columns of IGFs are first considered to be PP type. The contribution is estimated in terms of story shear. The shear ratio V_2, defined as V_I / V_T, is introduced for this purpose. For a given direction and story, V_I represents the shear resisted by all IGFs in that story and V_T represents the total shear resisted by the perimeter moment-resisting steel frames and the IGFs. This ratio is estimated for both horizontal directions.

Values of the V_2 ratio, in percentage, are shown in Figures 5(a) and 5(b) for Models 1 and 2, respectively, for the excitation in the N-S direction. It is observed that the V_2 values vary significantly from one model to another and from one story to another without showing any trend. The most important observation that can be made is that the values of V_2 are not negligible in many cases. Even though the relative stiffness of an IGF compared to that of a MRSF for the nonlinear time domain dynamics analysis depends on the time step being considered, the information on the static linear stiffness of the two systems is expected to give some information on the magnitude of the IGF contribution. The value of the (combined) lateral stiffness of all IGFs associated with the displacement of Floor 1 of Model 1 is estimated to be 275000 lb/in (4.8×10^7 N/m) by using the static linear analysis. The corresponding value for both MRSFs is 1650000 lb/in (2.9×10^8 N/m). Thus, the stiffness ratio is about 17%.

Values larger than 20% are observed for Story 1 of Model 1 in several cases. For the other stories the contribution of IGFs is smaller than 10% for most of the earthquake ground motions. The statistics of V_2 are summarized in Table 5. The uncertainty associated with the estimation, in terms of COV, is also found to be relatively high compared to the other parameters. Plots similar to Figure 5 were also developed for the excitation in the E-W direction but those are not shown here. The major observations made for the N-S direction are also valid for this direction. It is thus concluded that the contribution of IGFs to the lateral resistance could be significant and therefore should not be overlooked in the design of the structural systems under consideration.



(a) (b)

Fig. 5 V_2 values (in percentage) for (a) Model 1 and (b) Model 2 with PP connections, in N-S direction

2. Gravity Frames with SR Connections

The magnitude of the V_2 ratio is estimated while considering the realistic stiffness of the beam-to-column connection of the IGFs, as obtained by the PRCONN program. It is important to note that only sizes of the members are given in FEMA (2000); no information on the connections in the IGFs is given. To generate information on the four parameters in the Richard model using PRCONN, the connections are assumed to be typical "double web-angle connections". Thus, when the beam size is W16×26, the Richard parameters are estimated to be k = 16,400 kN-m, k_p = 851 kN-m, M_0 = 72 kN-m and N = 2. For the beam size of W18×35, the corresponding parameters are obtained as 28,800 kN-m, 1448 kN-m, 109 kN-m and 2, respectively. The maximum moment that the connections can transmit is observed to be about 0.25 of the plastic moment capacity M_P of the beam.

The results of V_2 ratio for Model 1 and E-W excitation are shown in Figure 6 for both PP and SR connections. As expected, the results clearly indicate that the contribution of IGFs to the lateral resistance increases when the stiffness of the connections is considered. This increase is particularly important for the upper stories. For example, for Story 3, the V_2 values are smaller than 10% in most of the cases for the frames with PP connections. Hence, there may not be any implication in the analysis and design of IGFs. However, when the connections are considered to be of SR type, the contribution of IGFs could be about 35%. This contribution is not small and cannot be ignored in the analysis and design of the members in IGFs. The major implication here is that the members in IGFs may not be able to carry these unexpected load effects in the case of non-negligible lateral loads.

Table 5: Statistics of V_2 Ratio

Model		Story	N-S Direction			E-W Direction		
			μ	σ	COV	μ	σ	COV
1	Elastic	1	18.8	4.5	0.24	14.3	1.2	0.08
		2	8.3	5.3	0.64	8.6	3.0	0.36
		3	9.4	5.1	0.54	7.9	3.6	0.46
	Inelastic	1	17.8	4.4	0.25	15.6	1.6	0.10
		2	8.4	4.2	0.50	8.6	3.1	0.35
		3	9.4	5.1	0.54	8.2	3.5	0.43
2	Elastic	2	16.4	0.9	0.05	24.4	1.2	0.05
		3	10.7	3.4	0.32	4.2	1.0	0.24
		4	2.3	1.6	0.70	3.8	1.0	0.26
		5	1.9	0.4	0.21	3.1	1.7	0.55
		6	3.0	1.3	0.43	4.7	2.0	0.43
		7	2.6	0.6	0.23	4.4	1.2	0.27
		8	2.9	1.2	0.41	5.3	2.1	0.40
		9	3.6	0.9	0.25	6.4	1.7	0.27
		10	3.8	1.2	0.32	6.4	1.8	0.28
	Inelastic	2	16.4	0.9	0.05	24.7	1.5	0.06
		3	3.5	0.7	0.20	4.2	1.1	0.26
		4	2.3	1.5	0.65	3.8	1.1	0.29
		5	1.9	0.4	0.21	3.1	1.7	0.55
		6	3.0	1.3	0.43	4.7	2.0	0.43
		7	2.6	0.6	0.23	4.4	1.2	0.27
		8	2.9	1.2	0.41	5.3	2.0	0.38
		9	3.6	0.9	0.25	6.4	1.7	0.27
		10	3.9	1.2	0.31	6.4	1.4	0.22

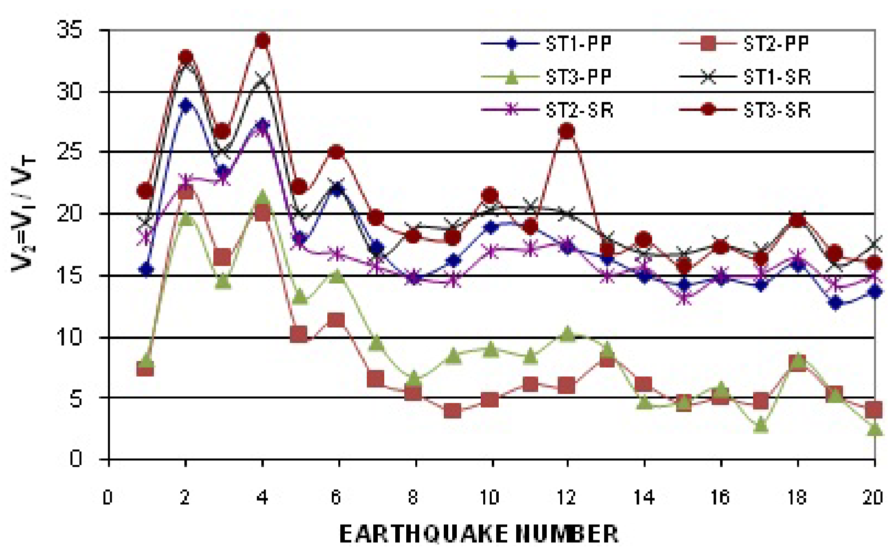
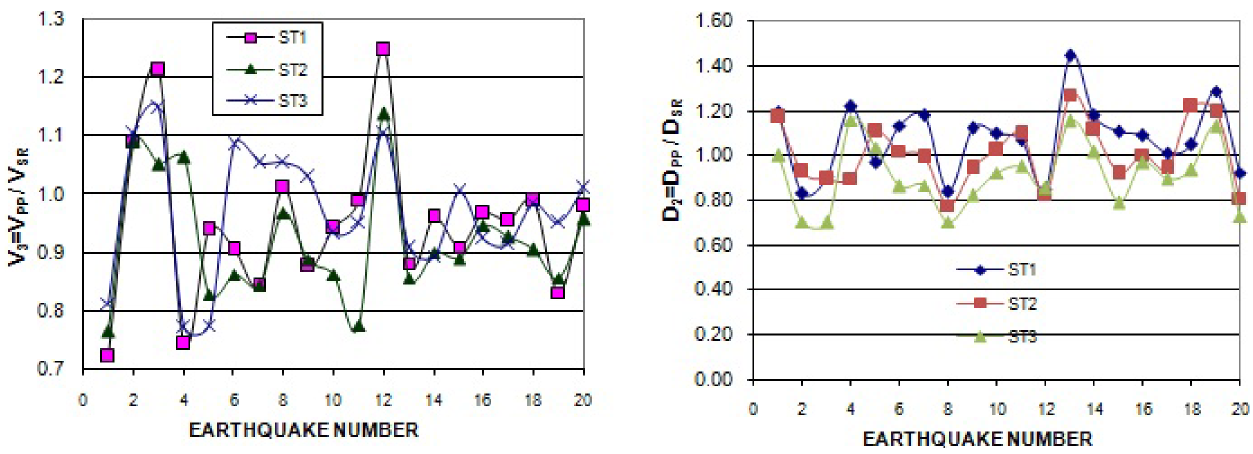


Fig. 6 V_2 values (in percentage) for Model 1 with PP and SR connections, in E-W direction

ISSUE 3—EFFECT ON STRUCTURAL RESPONSES OF THE CONNECTION STIFFNESS OF IGFs

The effect of the stiffness of the connections in IGFs on structural responses, in terms of interstory shears and displacements, and resultant stresses on individual members, is addressed in this part of the paper. Numerical results are presented only for Model 1 and E-W excitation. Interstory base shear is discussed first. The V_3 parameter, defined as V_{PP}/V_{SR}, is used for this purpose. For a given story, V_{PP} represents the shear in the story when PP connections are considered in the IGFs in the 3-D model. V_{SR} also represent the same response, except that the connections are considered to be of SR type. Results are shown in Figure 7(a). It is observed that the values of V_3 are smaller than unity in most cases, thus indicating that the interstory shears increase when the stiffness of the connections is considered appropriately. However, those can be larger than unity. The mean values of V_3 for the three stories are 0.93, 0.94 and 0.98, respectively. For the static application of lateral loads, the interstory shears are expected to increase if the stiffness of the connections is considered. However, this may not be true for the dynamic application of lateral loads (Reyes-Salazar and Haldar, 2001a, 2001b) since dynamic responses depend on many parameters not considered in static analyses. The dynamic properties (e.g., natural frequencies) are expected to be different for the frames with PP and SR connections, and the energy dissipation characteristics of SR connections are expected to have a significant effect on structural responses (Reyes-Salazar and Haldar, 2001b). It has been shown that the dissipated energy in SR connections may be comparable, and even larger, to that due to viscous damping and the hysteretic behaviour at plastic hinges (Reyes-Salazar and Haldar, 2001b). The variations in the natural frequencies of different structural arrangements, different energy dissipation characteristics and the frequency contents of ground motions will affect the responses as clearly illustrated in Figure 7. It may be seen that V_3 varies significantly from one earthquake ground motion to another, even though the maximum deformation is approximately same for all earthquake ground motions, with interstory drift being about 1%. The implication of this observation is that the seismic behavior of the frame with SR connections can be quite different from that of the frame with idealized PP connections.



(a) (b)

Fig. 7 (a) Shear ratio V_3 and (b) displacement ratio D_2, for PP and SR connections

Results for the interstory displacements are studied next. The D_2 ratio, defined as D_{PP}/D_{SR}, is used in this case. The notations D_{PP} and D_{SR} represent the average interstory displacements of the IGFs with PP and SR connections, respectively. The values of D_2 are shown in Figure 7(b). The D_2 values vary from one earthquake ground motion to another and from one story to another without showing any trend. However, these values are slightly greater than unity in most cases. The mean values for Story 1, Story 2 and Story 3 are 1.06, 1.01, and 0.94, respectively. This indicates that, on average, the dynamic displacements of the frames with PP connections are similar to those of the frames with SR connections. Similar observation was made in other studies (Reyes-Salazar and Haldar, 2001a, 2001b).

Results in terms of axial loads and moments on some columns at the base of the MRSFs are also estimated but are not shown here. It is observed that the values of these parameters may also decrease when the stiffness of the connections is considered appropriately, but the extent of this decrease is smaller than that for the interstory shear response.

CONCLUSIONS

To improve economy and increase efficiency in designing steel buildings made with W-shape members, MRSFs have become very popular. The underlying concept of such structural arrangements is that MRSFs are provided in the perimeter of the building to carry the seismic loads and IGFs are provided to carry only the gravity loads. Individually, each perimeter frame can be considered to behave in two dimensions in a three-dimensional structure, thus providing a simpler frame to analyze and design. Further, since fewer MRCs are used in such a structural configuration, the overall cost of structures is expected to be lower. FEMA (2000) suggested structural arrangements and member sizes of three such buildings and those can be considered as benchmark models. The behaviour of these model buildings under seismic loading, considering the implications of several modeling assumptions, has been critically evaluated in this paper. With the help of a sophisticated computer program developed by the authors, two of the model buildings were excited by twenty different recorded earthquake ground motions. Linear and nonlinear global and local responses were then calculated. Based on this study, it has been been concluded that the perimeter MRSFs with interior IGFs provide an economical alternative for steel structures to carry seismic loading. Modeling 3-D structures with 2-D perimeter frames in two orthogonal directions has been found to be conservative, considering both global and local response behavior. As expected, ignoring the lateral load carrying capacity of IGFs or the rigidity of PP connections in those adds several levels of conservatism in the overall design of a structure. This study has also indicated that if the connection rigidities are considered properly, the IGFs may need to carry up to 35% of the total lateral load, which is usually ignored in the analysis and design of members in those. The major implication is that the members in the IGFs may not be able to withstand the effects of this unexpected load in the case of non-negligible lateral loads. However, if the connection rigidities are considered properly, this will add complexity to the analysis of such frames, thus compromising basic simplicity, which is one of the desirable features of structures made with perimeter MRSFs and interior IGFs.

ACKNOWLEDGEMENTS

This paper is based on the work supported by El Consejo Nacional de Ciencia y Tecnología (CONACyT) under grant 50298-J and by La Universidad Autónoma de Sinaloa under grant PROFAPI 2008/153. Financial Support was also received from The University of Arizona and The Universidad Autónoma del Estado de México. 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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