12WCEE 2000

CONSIDERATION OF VERTICAL ACCELERATION AND FLEXIBILITY OF CONNECTIONS ON SEISMIC RESPONSE OF STEEL FRAMES

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SUMMARY

The influence of the vertical component of earthquakes and the flexibility of connections on the structural responses of moment-resisting steel frames (MRSFs) is evaluated. Specifically, this study addresses two major seismic design guidelines for buildings which consider the effect of the vertical acceleration of strong motions, namely, the National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Building, and the Mexico City Seismic Code. It is observed that, in general, both the NEHRP provisions and the Mexican Code reasonably estimate the maximum horizontal deflection at the top of the frame and bending moments at the lower level columns. However, both codes significantly underestimate the axial loads in columns. Results also indicate that even though frames are usually designed under the assumption of frames with fully restrained connections, they are essentially frames with partially restrained connections. The unintended modeling error introduces flexibility in the frames and may considerably increase the deformation of them.

INTRODUCTION

The influence of the vertical component of an earthquake and the flexibility of connections in moment-resisting steel frames (MRSFs) have long been of considerable interest to the profession. Several recorded ground accelerations of the Northridge Earthquake of 1994 indicate that the vertical component was much larger than what is usually considered normal in design. The damage associated with the Northridge earthquake prompted discussion of whether the excessive vertical acceleration may have caused the damage. Furthermore, the connections in steel frames are essentially partially restrained (PR), even though they are usually designed as fully restrained (FR). This reduces the stiffness of frames and may cause excessive deformation.

This study specifically addresses two major seismic design guidelines for buildings which consider the effect of the vertical acceleration, namely, the National Earthquake Hazard Reduction Program [NEHRP, 1994] Recommended Provisions for Seismic Regulations for New Building, and the Mexico City Seismic Code [CFE, 1993]. The design requirements for other codes are expected to be similar. The two codes have different design guidelines on how to consider the effect of the vertical component. Using nonlinear responses estimated as realistically as possible for three steel frames excited by thirteen actual time histories, most of them recorded during the Northridge earthquake, the two codes are compared.

The ratio of the peak ground acceleration of the vertical component (PGA_V) to the maximum horizontal peak ground acceleration (PGA_H), denoted hereafter as R, is used to study the influence of the vertical component on the overall seismic response of structures. For normal earthquakes, the R parameter is expected to be around 2/3. However, as shown in Table 1, for some time histories recorded during the Norhtridge Earthquake this ratio can be as large as 1.11. Thus, the Northridge Earthquake gives the profession an opportunity to evaluate the adequacy of the provisions suggested in design codes on how to consider the effect of the vertical component.

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The influence of the flexibility of connections on the overall nonlinear response of steel frames is also studied. Steel frames are usually analyzed assuming all the connections are FR type. However, considering the practical design aspects of connections, this is rarely true. This practice introduces unintended flexibility in the frame. To define the rigidity of the connection, a parameter called the *T* ratio is introduced. It is the ratio of the moment the connection would have to carry according to the beam line theory [Disque, 1964] and the fixed end moment of the beams. Since a *T* ratio of 1.0 representing an FR connection is rarely achieved in a typical steel frame, the connection is considered to be FR if *T* is at least 0.9. Thus, it is necessary to compare the seismic response, considering the idealistic assumption that the *T* ratio is exactly 1.0, to the seismic response of steel frames when the *T* ratio is 0.9. In this paper the Richard's model [Richard, 1993] is adopted to represent the behavior of the PR connections.

This comparative study is intended to establish the accuracy of current analysis procedures. Its success will depend on the evaluation of the nonlinear seismic response behavior of steel frames subjected to several strong motions. In this study a nonlinear time-domain finite element program developed by the authors and their associates [Gao and Haldar, 1995] is used to realistically evaluate the seismic responses by simultaneously applying the horizontal and vertical components and modeling the connection behavior as realistically as possible. The program estimates nonlinear seismic responses of steel frames with PR connections considering all major sources of energy dissipations.

THE NEHRP AND MEXICAN CODE PROVISIONS

In the 1994 edition of the NEHRP Provisions, a new requirement was added to consider the combined effects of the horizontal and vertical components on the structural response. It is addressed indirectly in the section on "Combination of Load Effects". It is suggested that the effect of gravity loads and seismic forces be combined in accordance with the factored load combinations as presented in the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures [ASCE 7-95], except that the effect of seismic loads, E, shall be defined as $E = Q_E + 0.5\, C_a\, D$ to consider the effect of both the horizontal and vertical components of an earthquake, where Q_E is the effect of horizontal seismic forces, C_a is the seismic coefficient based upon the soil profile type and the value of A_a as determined from Sec. 1.4.2.3 or Table 1.4.2.4a of the NEHRP Provisions, and D is the effect of the dead load. The commentary of the Provisions further adds that "0.5 C_a was placed on the dead load to account for the effects of vertical acceleration. The 0.5 C_a factor on dead load is not intended to represent the total vertical response. The concurrent maximum response of vertical acceleration and horizontal accelerations, direct and orthogonal, is unlikely and, therefore, the direct addition of responses was not considered appropriate."

In the Mexico City Seismic Code, the effect of the vertical component is considered to be a fraction of the effect of the largest horizontal component. It states that "For the buildings located in seismic zones C and D the effect of the vertical component should be considered. This effect shall be taken as 2/3 of that of the largest horizontal component. This effect while combined with gravity and horizontal component effects should be taken as 0.3 of the above equivalent vertical effect". Effectively, the code recommends that the effect of the vertical component should be estimated as 20 percent (the product of 2/3 and 0.3) of the effect of the largest horizontal component. This criterion can be interpreted another way. If the horizonal maximum response is H, then the vertical maximum response will be 2/3 H. Assuming that both maximums occurs at the same time and using the square root of the sum of squares (SRSS) rule, the total response considering both components can be calculated as $\sqrt{H^2 + (2/3\, H\,)^2} = 1.2\, H.$1 Obviously,the requirements in the Mexico City Seismic Code appear to be much more conservative than those of the Provisions because of this assumption.

MATHEMATICAL FORMULATION

An efficient finite element-based time-domain nonlinear analysis algorithm, already developed for the authors and their associates [Gao and Haldar, 1995; Reyes-Salazar, 1997] is used to estimate the effect of the vertical component and the flexibility of connections on the overall structural response. The procedure estimates nonlinear seismic responses of steel frames with PR connections considering all major sources of energy dissipation. Material and geometric nonlinearities are considered. Considering its efficiency, particularly for steel frame structures, the assumed stress-based finite element method [Kondoh and Atluri, 1987; Haldar and Nee, 1989; Shi and Atluri, 1988] is used. Using 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. Furthermore, information on material nonlinearity and connection flexibility can be incorporated in the algorithm without losing its basic simplicity. It gives very accurate results and is very efficient compared to the displacement-based approach. The procedure has been studied and extensively verified with existing theoretical and experimental results. Details of the algorithm are not given here because of lack of space.

DESCRIPTION OF STRUCTURES AND EARTHQUAKES

Three steel frame structures representing different dynamic characteristics, as shown in Fig. 1, are considered in this study. They are a three story, an eight story, and a fifteen story steel frames. They will be denoted hereafter as Frame 1, 2, and 3. They represent short, intermediate and tall buildings, respectively. The geometry of these three frames is shown in Fig. 1, and their member sizes are given in Table 2. The story height for these frames is a constant of 3.66 m, and the width of each bay is 7.32 m. In all these frames, the columns are assumed to be made of Grade-50 steel and the girders are made of A36 steel. In the seismic analysis of these frames, equivalent nodal forces were calculated, as required for the assumed stress-based finite element formulation used in this study. One node was placed at the mid-span of each of the girders. Each node is considered to have three degrees of freedom. The natural periods of the three frames are 0.31 sec, 0.54 sec, and 0.68 sec, respectively. Single Plate Top and Seat connections were used in all the frames. The stiffness of the connections are tuned up in such a way that their *T* ratio is 0.9.

These three frames with different dynamic characteristics are subjected to thirteen strong motion earthquakes. The first one is the El Centro Earthquake of 1940, and the other twelve are from the Northridge earthquake of 1994 recorded at the following stations: Los Angeles 1526 Edgemont Ave., Los Angeles Wadsworth VA, Los Angeles 10660 Wilshire Bvld., Los Angeles Griffith Observatory, Jenson Filtration Plant, Los Angeles Wadsworth VA, Topanga Fire Station, Sherman Oaks 1525, Ventura Blvd., Los Angeles 10751 Wilshire Blvd., Canoga Park Santa Susana, and Los Angeles 4939 Wilshire Blvd. The El Centro earthquake time histories are widely used in the profession to represent a typical earthquake, and are used here for the reference purposes only. The other twelve are strong motion time histories of the Northridge Earthquake of 1994, recorded at different locations. The peak ground accelerations of the earthquakes are given in Table 1. They are presented in increasing order for the ratio *R*. These earthquakes are denoted hereafter as Earthquakes 1 through 13.

EFFECT OF VERTICAL COMPONENT

Using the algorithm discussed earlier, the nonlinear response of the frames is estimated for 2% and 5% of the critical damping (ξ), in terms of the maximum lateral displacement at the top of the frame DMAX, and the maximum bending moments and axial loads at the ground level for the interior and exterior columns. All the frames are also subjected to static applications of the dead load as suggested in the NEHRP Provisions, and the corresponding responses are evaluated. For comparison purposes, the error term is defined as:

$$E = \frac{\text{Code Specified Value - Analytical Result}}{\text{Code Specified Value}} \tag{1}$$

Considering the dead load, earthquake load, and load combination suggested in ASCE 7-95, Eq. 1 can be rewritten for the NEHRP Provisions as:

$$E_{NEHRP} = \frac{(1.2\,D + H + 0.5\,C_a\,D) - (1.2\,D + HV)}{(1.2\,D + H + 0.5\,C_a\,D)} \tag{2}$$

where the term 1.2 D + H + 0.5 C_a D represents the combined effect of dead load, horizontal seismic load and vertical seismic load according to the NERHP Provisions; the term 1.2 D + HV represents the combined effect of dead load, horizontal and vertical seismic loads according to analytical results, H is the effect of the horizontal component containing the maximum PGA acting alone, 0.5 C_a D represents the effect of the vertical component, and HV represents the effect of both the horizontal and vertical components acting simultaneously.

Similarly, for the Mexican Code, Eq. 1 can be expressed as:

$$E_{MEX} = \frac{(1.2\,D + H + 0.2\,H) - (1.2\,D + HV)}{(1.2\,D + H + 0.2\,H)} \tag{3}$$

where the term 1.2 D + H + 0.2 H represents the combined effect of dead load, horizontal seismic load and the vertical seismic load according to the Mexican Code, and 0.2 H represents the effect of the vertical component.

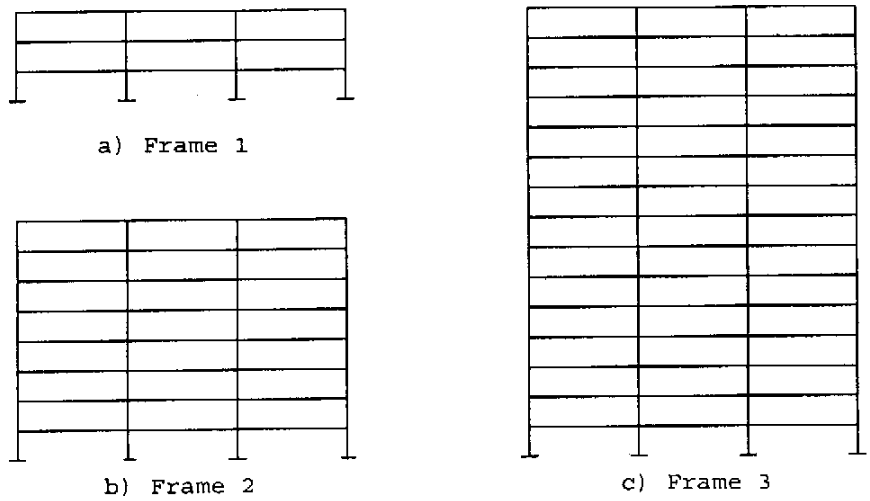

Figure 1. Three steel frames

Table 1. Earthquakes

EARTQ.		ACCELERATION (cm/sec^2)		
		PGA_V	PGA_H	R
1		206	342	0.60
NORTHRIDGE EARTHQUAKES	2	225	825	0.27
	3	135	359	0.38
	4	441	998	0.44
	5	142	276	0.51
	6	369	615	0.60
	7	152	250	0.61
	8	201	326	0.62
	9	377	551	0.68
	10	285	389	0.73
	11	326	380	0.86
	12	613	573	1.07
	13	526	472	1.11

Table 2. Member sizes

FRAME	STORY	EXT. COL.	INT. COL.	GIRDERS
1	1	W14x211	W14x283	W18x175
	2-3	W14x145	W14x211	W18x119
2	1-2	W14x370	W14x550	W24x335
	3-4	W14x277	W14x370	W24x279
	5-6	W14x211	W14x257	W24x192
	7-8	W14x193	W14x211	W24x131
3	1-4	W14x665	W14x730	W36x650
	5-6	W14x455	W14x665	W36x439
	7-8	W14x426	W14x455	W36x280
	9-10	W14x398	W14x426	W36x245
	10-12	W14x342	W14x398	W36x210
	13-15	W14x311	W14x342	W36x194

All other terms in Eq. 3 were defined earlier. A positive error in Eqs. 2 and 3 implies that the codes overestimate the load effect due to the vertical component; in other words, the codes' recommendations are conservative. A negative error indicates that the codes underestimate the load effect, and thus are unconservative. The responses of the three frames can be compared in light of the error terms just discussed.

Effect of the vertical component on DMAX and on bending moments

Results for Frame 1 are considered first. The DMAX values for the two damping ratios and all thirteen earthquakes are shown in Table 3. Column 3 contains analytical DMAX values for excitation by the horizontal component only. Column 4 contains the same information when the frame is subjected to 1.2 D plus both the horizontal and vertical components. Columns 5 and 6 contain the combined effect for the DMAX values according to the NEHRP Provisions and the Mexican Code, respectively. Using Eqs. 2 and 3, the corresponding error terms are calculated and are shown in Columns 7 and 8, respectively.

From the results given in Table 3, several important observations can be made. The maximum analytical horizontal deflections of the frame are observed to be almost the same for excitation by the horizontal component alone or by both the horizontal and vertical components. This is expected. Since the frame is symmetric and since it did not develop any plastic hinges, the effect of the vertical component in the estimation of the horizontal deflection is expected to be small. The DMAX values estimated according to the NEHRP Provisions (Column 5) are very similar to the analytical results. This is also expected, since the effect of the dead load on the DMAX calculation is negligible. The corresponding error according to Eq. 2 is also negligible (Column 7). However, the situation is quite different for the Mexican Code. The results in Column 6 indicate that the Mexican Code overestimates the DMAX values, and this overestimation is about 17% (Column 8). Thus, for Frame 1, the NEHRP Provisions estimate DMAX very accurately, but the Mexican Code overestimates it by about 17%.

Frames 2 and 3 are similarly analyzed. Results similar to Table 3 were also estimated for Frames 2 and 3. They can not be shown here due to lack of space. The major conclusions made for Frame 1 are also valid for these frames. The only additional observation is the heights of the frames or the *R* parameter, can not be correlated with the corresponding errors.

The effect of the vertical component on the evaluation of the bending moments for the interior and exterior columns at the ground is also estimated for the three frames. Results are not shown here due to lack of space. However, the major observations made for the DMAX calculations are also valid for the estimation of moments at the ground level of columns. The errors in the bending moment calculations according to the NEHRP Provisions are almost zero for both interior and exterior columns. The Mexican Code always overestimates the bending moments; the corresponding overestimation errors are about 17. As for the DMAX evaluation, these errors have no correlation with the *R* parameter or with the height of the frames.

Effect of the Vertical Component on Axial Loads in Columns.

The effect of the vertical component on the evaluation of the maximum axial loads at interior and exterior ground level columns for all the frames is considered next. The estimation errors according to both codes are calculated using Eqs. 2 and 3 for both interior and exterior columns. For ease of discussion, the errors versus *R* are plotted. Only underestimation of the axial load with errors larger than - 25%, which occurs for the interior column only, is emphasized in the following discussion. Other results cannot be shown due to lack of space. The results for the interior column of Frame 1 are shown in Fig. 2. Unlike the DMAX and the bending moment evaluation cases, the effect of the vertical component on the axial load estimation is observed to be significant. The underestimation error could be very large, on the order of - 50% for 2% damping according to the NEHRP Provisions, and about - 70% according to the Mexican Code. It is observed from the above figure that for a given code, the error is always larger for 2% damping than for 5% damping. It is also observed that the unconservative error associated with the Mexican Code is greater than that of the NERHP Provisions.

Frames 2 and 3 are similarly analyzed. The results are not shown here. The major observations made for Frame 1 apply to Frames 2 and 3. Although the NEHRP Provisions are better than the Mexican Code, the unconservative error associated with it may not be acceptable. Results indicate that the magnitude of the unconservative error is not a function of the height of the frame; however, it increases as the *R* value increases.

Table 3. Top displacements for Frame 1

EAR (1)	ξ (2)	H (cm) (3)	1.2D+HV (cm) (4)	NEHRP (cm) (5)	MEX (cm) (6)	E_{NEHRP} Eq. 2 (7)	E_{MEX} Eq. 3 (8)
1	2	2.88	2.86	2.88	3.46	1	17
	5	2.05	2.06	2.05	2.46	0	16
2	2	6.92	6.92	6.92	8.30	0	17
	5	5.38	5.37	5.38	6.46	0	17
3	2	1.82	1.82	1.82	2.18	0	17
	5	1.71	1.72	1.71	2.05	-1	16
4	2	6.39	6.40	6.39	7.67	0	17
	5	4.54	4.54	4.54	5.45	0	17
5	2	1.79	1.79	1.79	2.15	0	17
	5	1.49	1.48	1.49	1.79	1	17
6	2	3.79	3.78	3.79	4.55	0	17
	5	3.08	3.08	3.08	3.70	0	17
7	2	1.08	1.08	1.08	1.30	0	17
	5	0.81	0.82	0.81	0.97	-1	15
8	2	4.26	4.26	4.26	5.11	0	17
	5	2.65	2.66	2.65	3.18	0	16
9	2	7.44	7.46	7.44	8.93	0	16
	5	4.74	4.74	4.74	5.69	0	17
10	2	2.32	2.32	2.32	2.78	0	17
	5	1.78	1.78	1.78	2.14	0	17
11	2	1.96	1.96	1.96	2.35	0	17
	5	1.52	1.52	1.52	1.82	0	16
12	2	4.47	4.48	4.47	5.36	0	16
	5	3.59	3.59	3.59	4.31	0	17
13	2	2.74	2.74	2.74	3.29	0	17
	5	2.29	2.28	2.29	2.75	0	17

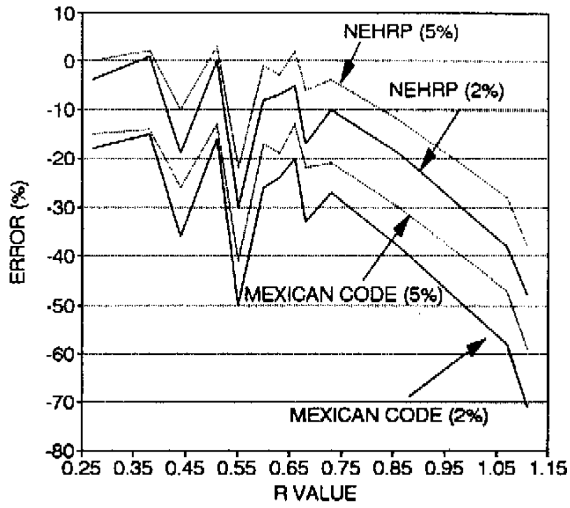

Figure 2. Error in the axial load on the interior colun of Frame 2

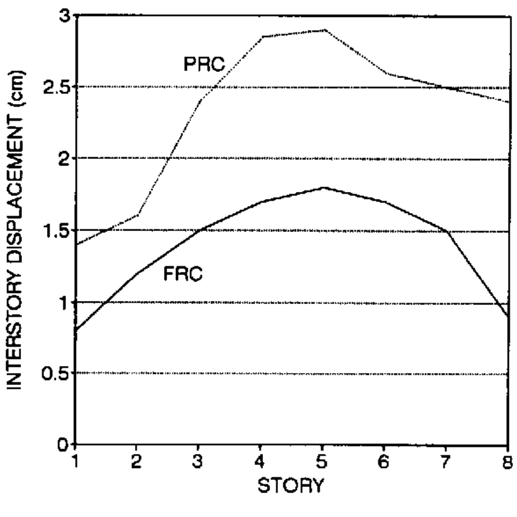

Figure 3. Frame 2 and Earthquake 1

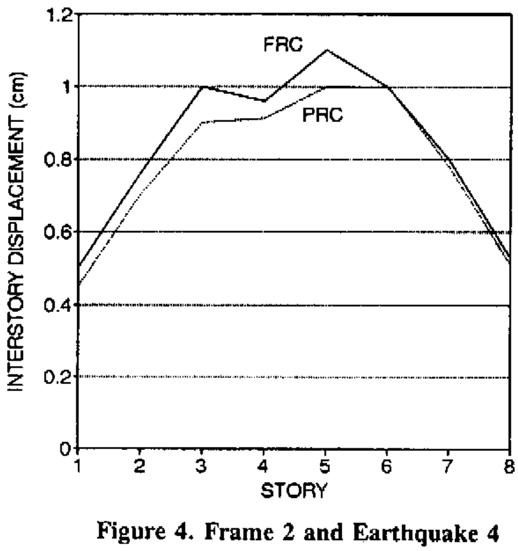

Figure 4. Frame 2 and Earthquake 4

EFFECT OF FLEXIBILITY OF CONNECTIONS

Results of the three frames subjected to all the thirteen earthquakes, in terms of the maximum interstory displacements, are evaluated for 2% and 5% of the critical damping. It needs to be pointed out that since the *T* ratio for all the connections is at least 0.9, using the standard design practice they can be assumed to be FR-type. However, realistically they are PR-type.

The interstory displacements for Frame 2 and Earthquake 1 are shown in Fig. 3 for 2 % damping, representing a typical case. From this figure, it is observed that the interstory displacements for the frame with PR connections are much larger than the corresponding displacements for the frame with FR connections. This is the case for many of the earthquakes, particularly for 2% damping. For some other cases, however, the interstory displacements can be smaller for the frame with PR connections than for the frame with FR connections, as shown in Fig. 4 for Frame 2, Earthquake 4 and 2% damping.

It is not possible to show the interstory displacements for all the three frames excited by all the earthquakes due to lack of space. However, results indicate that the flexibility of connections may increase significantly the deformation of the frame. Thus, the assumption of FR connection for a connection with a T ratio of at least 0.9, may be an unconservative assumption, indicating that the real flexibility of connections have to be considered in an appropriate way for any seismic or dynamic analysis.

CONCLUSIONS

The influence of the vertical component of earthquakes and the flexibility of connections on the structural responses of moment-resisting steel frames (MRSFs) is studied. Using a time domain nonlinear finite element program developed by the authors, the seismic response of steel frames is evaluated by applying the horizontal and vertical components of earthquake motion simultaneously and modeling the flexibility of connections as realistically as possible. Two major seismic design guidelines for buildings which consider the effect of the vertical acceleration of strong motions, namely, the National Earthquake Hazard Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Building, and the Mexico City Seismic Code, are reevaluated. It is observed that, in general, both the NEHRP provisions and the Mexican Code reasonably estimate the maximum horizontal deflection at the top of the frame and bending moments at the bottom columns. However, both codes significantly underestimate the axial loads in columns.

Although the effect of the vertical component in the moment calculation of columns is negligible or conservative in most cases, it may increase the axial load significantly. Since they are designed as beam-columns, the increase in the axial load will have very a detrimental effect on the performance of the columns. In light of the results obtained in this study, the design requirements for the vertical components, as outlined in the NEHRP Provisions and the Mexican Code, need modification. At the very least, further study is required. Results also indicate that even though frames are usually designed under the assumption of frames with fully restrained connections, they are essentially frames with partially restrained connections. The unintended introduced flexibility may considerably increase the deformation of the frames. Thus, the assumption of FR connection for a connection with a T ratio of 0.9, may be an unconservative assumption, indicating that the real flexibility of connections have to be considered in an appropriate way.

ACKNOWLEDGEMENTS

This paper is based on work partly supported by the National Science Foundation under Grants No. MSM-8896267 and CMS-9526809. The financial support received from the American Institute of Steel Construction (AISC), Chicago, is appreciated. The work is also partially supported by El Consejo Nacional de Ciencia y Tecnologia (CONACyT) under grant 486100-5-28464U and by La Universidad Autonoma de Sinaloa (UAS), 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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NOTATION

DMAX	=maximum absolute top displacement.
ξ	=percent of critical damping.
D	=effect of dead load.
E	=effect of seismic load.
FR	=fully restrained.
MRSFs	=moment resisting steel frames.
PR	=partially restrained.
PGA_H	=horizontal peak ground acceleration.
PGA_V	=vertical peak ground acceleration.
R	=the ratio of PGA_V to PGA_H.
T	=the ratio of the connection moment and the fixed end moment.