The 2013 World Congress on
Advances in Structural Engineering and Mechanics (ASEM13)
Jeju, Korea, September 8-12, 2013

Seismic response reduction due to viscous damping and material yielding for 3D steel buildings with PMRF

[1)]Alfredo Reyes-Salazar[1)], Ramón E. Rodelo-López[2)], Eden Bojórquez[1)], J. Luz Rivera-Salas and Juan I. Velazquez-Dimas[1)]

[1), 2)]*Facultad de Ingeniería, Universidad Autónoma de Sinaloa, Ciudad Universitaria, Culiacán, Sinaloa, México, *reyes@uas.edu.mx*

Achintya Haldar[3)]

[3)]*Department of Civil Engineering and Engineering Mechanics, University of Arizona, Tucson, Arizona. USA*

ABSTRACT

The evaluation of the effect of viscous damping and yielding of the material on the reduction of the seismic responses of steel buildings with perimeter moment resisting frames, modeled as three-dimensional (3D) complex multi-degree of freedom (MDOF) systems, constitutes the main objective of this paper. The results are compared with those of equivalent 3D structural representations with spatial moment resisting frames as well as with those of bi-dimensional and equivalent single degree of freedom idealizations. The results indicate that the reduction significantly vary from one earthquake to another, even thought the earthquakes were normalized with respect to the pseudo acceleration evaluated at the fundamental structural period ($S_a(T_1)$), reflecting the influence of the earthquake frequency contents and the contribution of several modes on the structural responses. It is also observed that the reduction produced by damping may be larger or smaller than that of yielding. This reduction can significantly vary from one structural representation to another, and is smaller for global than for local response parameters, which in turn depends on the particular local response parameter and the location of the structural element under consideration. The uncertainty in the estimation is significantly larger for local response parameter and decreases as damping increases. It is concluded that, estimating the effect of damping and yielding on the seismic response of steel buildings by using simplified models may be a very crude approximation. Moreover, the effect of yielding should be explicitly calculated by using complex 3D MDOF models instead of estimating it in terms of equivalent viscous damping.

1. INTRODUCTION

Because of our limited knowledge about the *Earthquake Phenomenon*, seismic

[1), 3), 4), 5), 6)] Professor
[2)] Graduate Student
Note: Copied from the manuscript submitted to "Steel and Composite Structures, An International Journal" for presentation at ASEM13 Congress

analysis and design procedures for structures are updated or modified on a continuous basis. Several methods with different degrees of sophistication have been suggested in most codes. They include the static equivalent lateral force (SELF) procedure, the nonlinear static procedure (PUSHOVER), and several types of dynamic analysis procedures like modal response, spectral, linear time-history, and nonlinear time-history analyses. Even though in current building codes the inelastic behavior of structures is explicitly considered by using nonlinear methods, shifting away from the traditional elastic analysis, the use of simplified methods like SELF procedure are still broadly used. Many seismic building codes around the world permit the use of this procedure for regular structures with relative short periods.

According to the SELF procedure, buildings are designed to resist seismic equivalent static lateral forces which are related to the seismicity of the region and the type of structure under consideration. Some equations are given to estimate the base shear and the distribution of lateral forces over the height of the building. Static analysis of the building acted upon these forces provides the design forces. In the procedure, the elastic base shear is reduced by using a factor, here called seismic reduction factor (R) which mainly depends on the structural overstrength and the energy dissipation capacity which in turn depends on the structural system, structural material and level of detailing. This is particularly important for steel structures since energy dissipation is supposed to come from different sources.

Many mechanisms contribute to the energy dissipation in actual building structures. As it will be additionally discussed in the following sections of the paper, they have an important effect on the structural responses. In the case of seismic analysis of steel buildings, this dissipation is usually considered in two ways. An equivalent viscous damper is used to model the energy dissipation at deformations within the elastic limit of the structure while the dissipated energy due to inelastic behavior (yielding) of the material is considered by including the inelastic relationship between resisting forces and deformation. The effect of the energy dissipated by each of these mechanisms on the structural response has been studied for simplified structural systems but not for complex structural representations. The evaluation of the effect of damping and yielding on the seismic response of 3D steel buildings with perimeter moment resisting frames, modeled as complex multi-degree of freedom (MDOF) systems, as will be discussed below in more detail, constitute the primary objective of this study.

2. LITERATURE REVIEW

There have been many investigations regarding the estimation of the dissipated energy as well as its effect on the seismic response of steel buildings and the related force reduction and ductility factors. One of the first investigations was conducted by Newmark and Hall (1982). They proposed an approximated procedure for constructing the inelastic response spectra from the basic elastic design spectra by relating the seismic reduction and the ductility parameters. Because of its importance, this work was taken as a reference in many others investigations. Hadjian (1989) studied the reduction of the spectral accelerations to account for the inelastic behavior of structures. Nassar and Krawinkler (1991) studied the relationship between force reduction factors and ductility for SDOF and simplified (three-story single-bay) MDOF systems. Miranda and Bertero

(1994) proposed simplified expressions to estimate the inelastic design spectra as a function of the maximum tolerable ductility, the period of the system and the soil conditions of the site. Shen and Akbas (1999) proposed simplified expressions to estimate the input energy and the damping energy of steel moment resisting frames subjected to a group of ground motions recorded on different types of soils. They concluded that the energy concept based on SDOF systems has limitations when extended to realistic structural systems for design purposes. Reyes-Salazar and Haldar (2001a), by using simplified plane models found that the dissipation of energy produced by viscous damping, or by yielding of the material, is comparable to that of partially restrained connections. Reyes-Salazar (2002) studied the ductility capacity of plane steel moment resisting frames; local, story and global ductility were considered. It was shown that using SDOF systems to estimate the ductility capacity may be a very crude approximation. Arroyo-Espinoza and Terán-Gilmore (2003) from the study of the dynamic response of SDOF systems proposed expressions to estimate strength reduction factors that should be used to reduce the elastic response spectra to establish the design seismic forces for structures with different combinations of plastic and viscous energy dissipating capacities. Hong and Jian (2004) studied the impact of the uncertainty in the natural vibration period and damping ratio on the peak displacement of linear elastic and elasto-plastic SDOF systems. Karmakar and Gupta (2006) performed a parametric study to estimate the dependence of strength reduction factors on strong motion duration, earthquake magnitude, geological site conditions, and epicentral distance for elasto-plastic oscillators. Chopra (2007) studied the force reduction factors for MDOF systems modeled as shear buildings and its corresponding equivalent SDOF systems. The relative effect of yielding and damping for SDOF systems was also studied. It was shown that the effects of yielding should not been considered in terms of a fixed amount of equivalent viscous damping.

More recently, Ayoub and Chenouda (2009) developed response spectra plots for inelastic degrading structural systems subjected to seismic excitations. They proposed constitutive models for degrading structures which were calibrated against experimental data. Sanchez-Ricart (2010) reviewed the backgrounds that support the values of the reduction factor in the United States, Europe and Japan. It was concluded that the design reduction factor cannot be deduced directly from the performance of the buildings after real earthquakes since the performance implicitly includes the design structural overstrength and therefore, the structural overstrength must be quantified and excluded when calibrating the design reduction factor from the performance of the buildings after real earthquakes. Ceylan et al. (2010) estimated the strength reduction factor for prefabricated industrial structures having a single storey, one and two bays. Ganjavi and Hao (2012) studied the seismic response of linear and nonlinear MDOF systems subjected to a group of earthquakes recorded on alluvium and soft soils, considering different shear strength and stiffness distribution patterns. They showed that depending on the level of inelasticity, soil flexibility and number of degrees-of-freedoms (DOFs), structural characteristics distribution can significantly affect the strength demand and ductility reduction factor of MDOF systems.

In spite of the important contributions of the previous studies on the evaluation of the effects of energy dissipation, most of them were limited to SDOF systems, plane shear buildings or plane moment resisting steel frames. Inelastic behavior and energy dissipation of the structural elements existing in actual three-dimensional systems are not been considered. Reyes-Salazar and Haldar (1999, 2000, 2001a, 2001b) and Bojorquez et al (2010) found that moment resisting steel plane frames are very efficient in dissipating earthquake-induced energy and that the dissipated energy has an important effect on the structural response. Reyes-Salazar (2002) showed that the values of strength reduction factors depend on the amount of dissipated energy, which in turn depends on the plastic mechanism formed in the frames as well as on the loading, unloading and reloading process at plastic hinges. Moreover, it is important to emphasize that modeling structures as plane frames may not represent their actual behavior since the participation of some elements are not considered and the contribution of some vibration modes are ignored. The dynamic properties in terms of stiffness, mass distribution, natural frequencies and energy dissipation characteristics are expected to be different for SDOF, two-dimensional (2D) and three-dimensional (3D) modeling of such structures. The corresponding structural responses are also expected to be different. Due to advancement in the computer technology, the computational capabilities have significantly increased in the recent years. It is now possible to estimate the seismic response behavior by modeling structures in three dimensions as complex MDOF systems with thousand of degrees of freedoms and applying the seismic loadings in time domain as realistically as possible. Responses obtained in this way may represent the best estimate of the seismic responses. The accuracy of estimating the effect of the energy dissipated by damping or by yielding of the material on the global and local response parameters by using simplified SDOF or simplified MDOF systems can then be judged by comparing the results with those obtained from the complex 3D formulation.

3. OBJECTIVES

The specific objectives addressed in this study are:

Objective 1. Estimate the effect of damping on the seismic responses of steel buildings with MRSF modeled as 3D systems and compare them with those of the corresponding 2D and SDOF structures. Two cases of 3D models will be considered: a) with perimeter moment resisting frames (PMRF), and b) with spatial moment resisting frames (SMRF). The seismic responses are obtained in terms of global (interstory base shear and displacements) and local (axial load and bending moment) parameters. No yielding is allowed to occur in the models.

Objective 2. Estimate the effect of yielding on the seismic responses of steel buildings with MRSF modeled as 3D systems and compare them with those of the corresponding 2D and SDOF structures.

To reach the objectives of the study, the seismic responses of some structural models are estimated as accurately as possible by using three-dimensional time history analysis. The models are excited by several time histories recorded at hard and intermediate soils which were selected to represent the different characteristics of strong motions. Energy dissipation and higher mode contributions are explicitly considered. The used earthquakes are scaled in terms of spectral acceleration in the fundamental mode of vibration of the structure ($S_a(T_1)$) in such a way that for the critical earthquake the models develop a collapse mechanism or an interstory displacement of about 1.8 %.

4. MATHEMATICAL FORMULATION

To satisfy the objectives of the study, the nonlinear seismic responses of the steel buildings under consideration modeled as 3D complex MDOF structures are needed. An assumed stress-based finite element algorithm, developed and implemented by the authors and their associates (Gao and Haldar, 1995, Reyes-Salazar 1997) in a computer program, is used to estimate the responses. 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. The procedure and the algorithm have been extensively verified using available theoretical and experimental results (Reyes-Salazar and Haldar 2001a, Reyes-Salazar and Haldar 2001b).

The geometric and material nonlinearities are considered in the tangent stiffness matrix. The mathematical details of the derivation are not shown here, but can be found in the literature (Kondo and Atluri 1987). The material is considered to be linear elastic except at plastic hinges. Concentrated plasticity behavior is assumed at plastic hinge locations. In the past, several analytical procedures were proposed to predict the deformation of elasto-plastic frames under increasing seismic and static loads. However, most of these formulations were based on small deformation theory. In this study, each elasto-plastic beam-column element can experience arbitrary large rigid deformations and small relative deformations. Thus, in addition to the elastic stress-strain relationships, the plastic stress-strain relationships need to be incorporated into the constitutive equations if the yield condition is satisfied. Several yield criteria have been proposed in the literature in terms of stress components or nodal forces. Since the nodal forces can be obtained directly from the proposed method, the yield criteria used here is expressed in terms of nodal forces. When the combined action of the nodal forces satisfies a prescribed yield function at a given end of an element, a plastic hinge is assumed to occur instantaneously at that location. Plastic hinges are considered to form at the ends of the beam-columns elements. The yield function depends on both, the type of section and loading acting on the beam-column element (Mahadevan and Haldar 1991). The yield function for three-dimensional beam-column elements has the following general form:

$$f\,(P, M_x, M_y, M_z, \sigma_y) = 0 \quad at\ X = l_p \tag{1}$$

where P is the axial force, M_x *and* M_y are the acting bending moments with respect to the mayor and minor axis, respectively, M_z is the torsional moment, σ_y is the yield stress, and l_p is the location of the plastic hinge. For the W-type sections used in the models of this study, this equation has the following particular form:

$$\left(\frac{P}{P_n}\right)^2 + \left(\frac{M_x}{M_{nx}}\right)^2 + \left(\frac{M_y}{M_{ny}}\right)^2 + \left(\frac{M_z}{M_{nz}}\right)^2 - 1 = 0 \tag{2}$$

where P_n is the axial strength, M_{nx} and M_{ny} are the flexural strength with respect to the major and minor axis, respectively and M_{nz} is the torsional strength.

The additional axial deformations and relative rotations produced by the presence of plastic hinges are taken into account in the stiffness matrix and the internal force vector of the plastic stage. Explicit expressions for the elasto-plastic tangent stiffness matrix and the elasto-plastic internal force vector are also developed. The mathematical derivations can be found in the literature (Kondo and Atluri 1987). Depending on the level of earthquake excitation, in a typical structure, all the elements may remain elastic, or some of the elements will remain elastic and the rest will yield. The structural stiffness matrix and the internal force vector can be explicitly developed from the individual elements and their particular state (elastic or plastic).

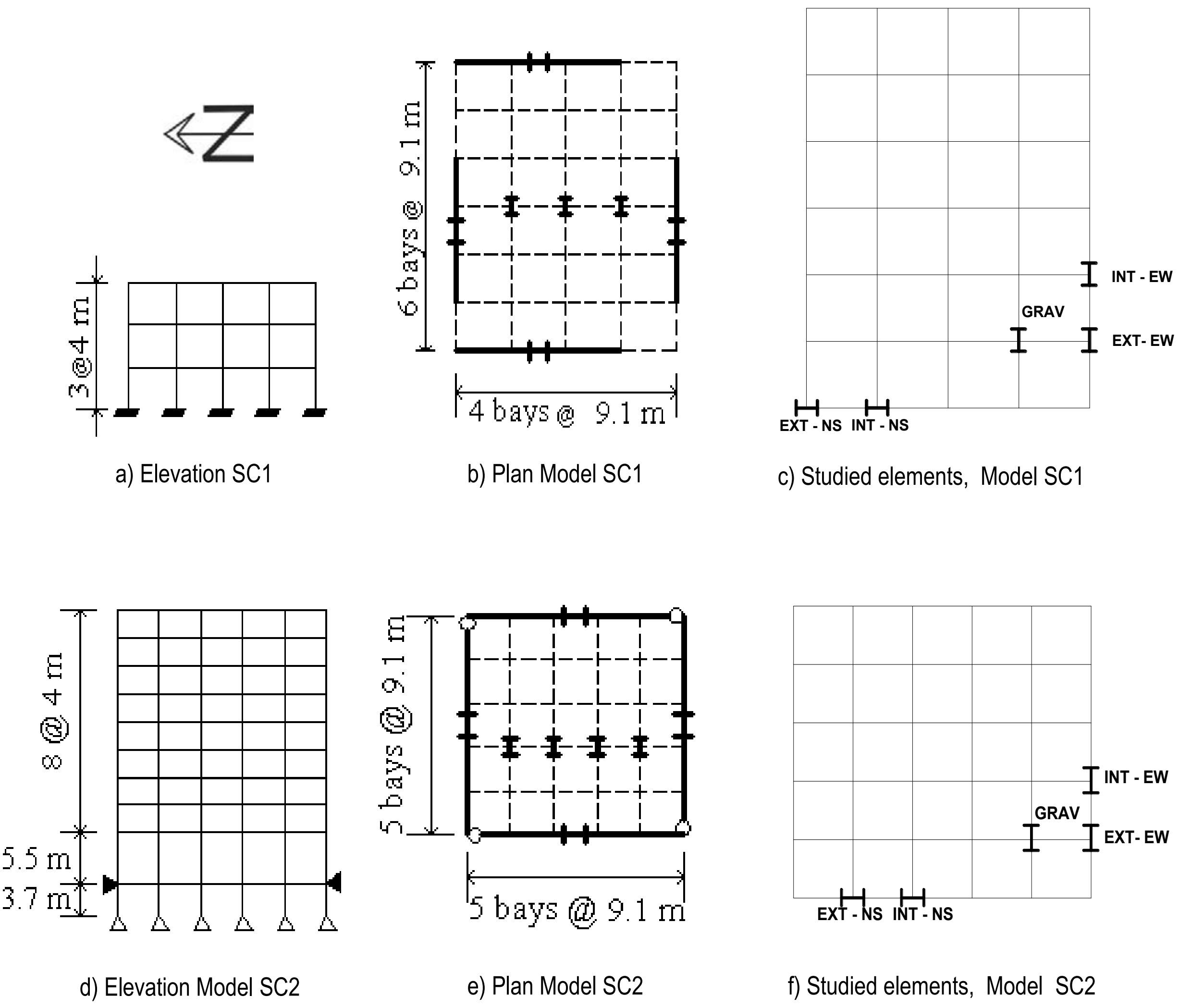
Based on an extensive literature review, it is observed that viscous Rayleigh-type damping is commonly used in the profession and is used in this study (Clough and Penzien 1993). The consideration of both the tangent stiffness and the mass matrices is a rational approach to estimate the energy dissipated by viscous damping in a nonlinear seismic analysis. The mass matrix is assumed to be concentrated-type. The step-by-step direct integration numerical analysis procedure and the Newmark β method (Bathe 1982) are used to solve the nonlinear seismic governing equation of the problem. A computer program has been developed to implement the solution procedure. The program was extensively verified using information available in the literature. The structural response behavior in terms of members' forces (axial load, shear force and bending moment), total base shear and interstory displacements, can be estimated using this computer program.

5. STRUCTURAL MODELS

5.1 3D buildings with PMRF (SAC Models)

As part of the SAC steel project (FEMA 2000), several steel model buildings were designed by three consulting firms. They considered 3-, 10- and 22- level buildings. These buildings are supposed to satisfy all code requirements existed at the time of the project development for the following three cities: Los Angeles (Uniform Building Code, 1997), Seattle (Uniform Building Code, 1997) 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 for numerical evaluations to address the issues discussed earlier. They will be denoted hereafter as Model SC1 and SC2, respectively and, in general, they will be referred as the SAC Models. These models have been used in many investigations.

The elevations of the models are given in Figs. 1a and 1d and their plans are given in Figs. 1b and 1e, respectively. The fundamental periods of Model SC1 and SC2 are estimated to be 1.02 and 2.34 sec. respectively. The 10-level building has a single-level basement. The columns of the PMRF of Model SC1 are fixed at the base while those of Model SC2 are pinned, as considered in the FEMA report. In all these frames, the columns are made of steel Grade-50 and the girders are of A36 steel. For both models, the columns in the Gravity Frames (GF) are considered to be pinned at the base. All the columns in PMRF bend about the strong axis and the strong axes of the gravity columns are oriented in the *N-S* direction, as indicated in Figs. 1b and 1e. The particular elements to study the response in terms of local responses parameters are given in Figs. 1c and 1f for Models SC1 and SC2, respectively. In these figures, the PMRF are represented by continuous lines while the interior GF are represented by dashed lines. For Model SC2, the PMRF meet at a corner. In this case, the beam-to-column connections are considered to be pinned to eliminate weak axis bending (Fig. 1e). As it can be seen, the buildings are essentially symmetrical in plan, thus no significant torsional moments are expected to occur. Sizes of beams and columns, as reported (FEMA 2000), are given in Table 1 for the two models. The designs of the PMRF in the two orthogonal directions were practically the same. Additional information for the models can be obtained from the FEMA report.



a) Elevation SC1 b) Plan Model SC1 c) Studied elements, Model SC1

d) Elevation Model SC2 e) Plan Model SC2 f) Studied elements, Model SC2

Figure 1. Elevation, plan and element location for Models SC1 and SC2

Table 1. Beam and columns sections for the SAC models

MODEL	MOMENT RESISTING FRAMES			GRAVITY FRAMES	
		COLUMNS	GIRDERS	COLUMNS	

	STORY	EXTERIOR	INTERIOR		BELOW PENTHOUSE	OTHERS	BEAMS
1	1\2	W14x257	W14x311	W33X118	W33X118	W14x68	W18x35
	2\3	W14x257	W14x312	W30X116	W30X116	W14x68	W18x35
	3\Roof	W14x257	W14x313	W24X68	W24X68	W14x68	W16x26
2	-1/1	W14x370	W14x500	W36x160	W36x160	W14x193	W18x44
	1/2	W14x370	W14x500	W36x160	W36x160	W14x193	W18x35
	2/3	W14x370	W14x500,W14x455	W36x160	W36x160	W14x193,W14x145	W18x35
	3/4	W14x370	W14x455	W36x135	W36x135	W14x145	W18x35
	4/5	W14x370,W14x283	W14x455,W14x370	W36x135	W36x135	W14x145,W14x109	W18x35
	5/6	W14x283	W14x370	W36x135	W36x135	W14x109	W18x35
	6/7	W14x283,W14x257	W14x370,W14x283	W36x135	W36x135	W14x109,W14x82	W18x35
	7/8	W14x257	W14x283	W30x99	W30x99	W14x82	W18x35
	8/9	W14x257,W14x233	W14x283,W14x257	W27x84	W27x84	W14x82,W14x48	W18x35
	9/Roof	W14x233	W14x257	W24x68	W24x68	W14x48	W16x26

The buildings are modeled as complex MDOF systems. Each column is represented by one element and each girder of the PMRF is represented by two elements, having a node at the mid-span. The slab is modeled by near-rigid struts, as considered in the FEMA study. Each node is considered to have six degrees of freedom when the buildings are modeled in three dimensions.

5.2 3D buildings with SMRF (EQ Models)

Because of economical considerations and the fragility of weak-axis connections, the standard practice during the recent past (after the 80s) in USA has been to build steel buildings with fully restrained connections (FRC) only on two frame lines in each direction. The redundancy of the buildings, however, is tremendously reduced. In Mexico, it is common to use steel buildings with FRC at the perimeter and the interior, in both horizontal directions. Due to the large number of FRC of this system, its redundancy is expected to be greater than those of the systems with only PMRF although the structural analysis is more complicated. Comparison of the performance of these two structural systems under the action of severe seismic loads, in terms of the effect of energy dissipated by damping and yielding, is undoubtedly of great interest to the profession and therefore it is addressed in this study. Equivalent models with SMRF are considered for this purpose. The equivalent models are designed in such a way that their elastic fundamental period, total mass, yield strength and lateral stiffness are fairly the same as those of the corresponding buildings with PMRF.

The member properties of the equivalent buildings are selected for one direction, say the *N-S* directions, and then in order to keep the equivalence, the same properties are assigned to the other direction. They are selected by considering the beam and column properties of the PMRF oriented in the direction under consideration, in addition to those of the beams and columns of the perpendicular PMRF. It must be noted that the columns of the later frames bend with respect to their minor axis. The ratio of moments of inertia, or plastic moments, between beams and columns was tried to keep as close as possible for the two structural systems. The same was considered for the case of interior and exterior columns. The equivalent models are referred, in particular, as Models EQ1 and EQ2 for the 3- and 10-level buildings, respectively, and, in general, as EQ Models. The resulting sections are shown in Table 2.

5.3. 2D models

For seismic analysis and design purposes, steel buildings with PMRF are modeled as plane frames. In this process, it is assumed that, for a given horizontal direction, half of the seismic loading is supported by the two PMRF oriented in that direction. However, as stated earlier, modeling 3D buildings as plane frames may not represent the actual behavior of the structure since the participation of some elements are not considered. Moreover, the stiffness and the dynamic properties in terms of natural frequencies, damping or energy dissipation characteristics, are expected to be different for two-dimensional and three-dimensional modeling of such structures. Thus, it will be of interest to estimate the relative effect of damping and yielding on the seismic response of steel buildings with PMRF, modeled as 3D structural systems and compare it with that of the structures modeled as 2D systems. These models will be denoted as Models 2D1 and 2D2 for the 3- and 10-level models, respectively, and, in general as 2D models.

Table 2. Beam and columns sections for the equivalent (EQ) Models

MODEL	STORY	COLUMNS		GIRDERS
		EXTERIOR	INTERIOR	
3-LEVEL	1\2	W16 X 67	W14 X 109	W12 X 170
	2\3	W16 X 67	W14 X 109	W14 X 120
	3\Roof	W16 X 67	W14 X 109	W16 X 40
10-LEVEL	-1/1	W18 X 143	W21 X 166	W24 X 162
	1/2	W18 X 143	W21 X 166	W24 X 162
	2/3	W18 X 143	W21 X 166	W24 X 162
	3/4	W18 X 143	W21 X 147	W21 X 166
	4/5	W18 X 143	W21 X 147	W21 X 166
	5/6	W21 X 93	W27 X 84	W21 X 166
	6/7	W21 X 93	W27 X 84	W21 X 166
	7/8	W14 X 145	W18 X 106	W24 X 68
	8/9	W14 X 145	W18 X 106	W12 X 152
	9/Roof	W24 X 62	W18 X 97	W16 X 67

5.4 SDOF Models

The relative effect of damping and yielding is also studied for *equivalent* single degree of freedom (SDOF) systems. One equivalent SDOF model is considered for the 3- and 10-level buildings. They will be particularly denoted hereafter as Models SD1 and SD2, respectively, and as SDF models in general. These systems have a SDOF in each

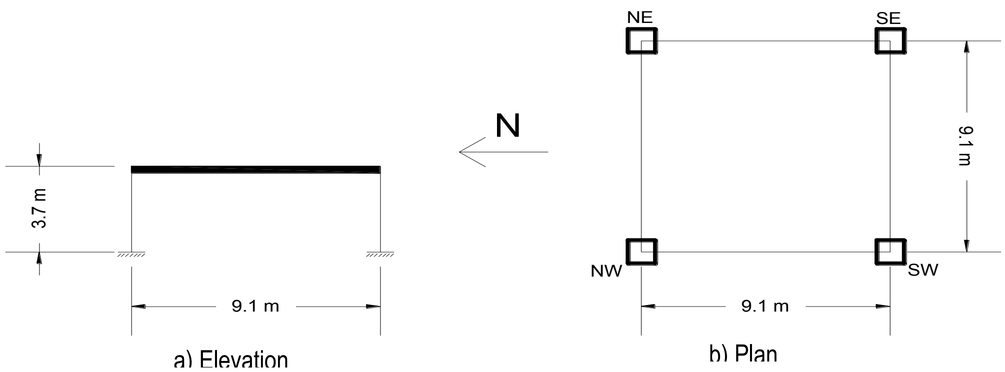


Figura 2. Elevation and plan of the equivalent SDF models

horizontal direction. The elevation and plan of these systems are shown in Fig. 2. The weight of the *equivalent* SDOF system is the same as the total weight of its corresponding MDOF system and its lateral stiffness is selected in such a way that its natural period is the same as the fundamental natural period of its corresponding MDOF system. In order to have the equivalence in both horizontal directions, box columns are used. The damping ratio and the yielding strength are selected to be the same for the SAC and the SDF models. The later was determined from a pushover analysis. It must be noted that in a strict sense, the simpler models are not the typical SDOF systems studied in the structural dynamics textbooks since axial forces can be developed in the columns under the action of horizontal excitations.

5.5 Earthquake loading

Dynamic responses of a structure excited by different earthquake time histories, even when they are normalized in terms of $Sa(T_1)$ or in terms of the peak ground acceleration, are expected to be different, reflecting their different frequency contents. Thus, evaluating structural responses excited by an earthquake may not reflect the behavior properly. To study the responses of the models comprehensively and to make meaningful conclusions, they are excited by twenty recorded earthquake motions in time domain with different frequency contents, recorded at different locations. As stated earlier the earthquake records are scaled in terms of spectral acceleration in the fundamental mode of vibration of the structure ($S_a(T_1)$) in such a way that for the critical earthquake the models develop a collapse mechanism or a maximum interstory displacement of about 1.8% (whatever occurs first). The characteristics of these earthquake time histories are given in Table 3. As shown in the table, the predominant periods of the earthquakes vary from 0.12 to 0.88 sec. The predominant period for each earthquake is defined as the period where the largest peak in the elastic response spectrum occurs, in terms of pseudo accelerations. The earthquake time histories were obtained from the Data Sets of the National Strong Motion Program (NSMP) of the United States Geological Surveys (USGS). Additional information on these earthquakes can be obtained from these data base.

Table 3. Earthquake Models

NUMBER	DATE	STATION	T (seg)	EPICENTER (km)	DEPTH (km)	MAGNITUDE	PGA (cm/s²)
1	09/06/80	Cerro Prieto	0.12	20	4.6	6.3	308
2	02/09/07	Lake MathewsDam	0.15	13	12.5	4.7	507
3	02/09/05	Salton Sea WildlifeRefuge	0.19	2	9.7	4.8	236
4	27/08/11	Bear Valley, WebbResidence	0.21	13	7.6	4.6	239
5	06/04/12	Paicines, HainHomestead	0.23	3.8	5.6	4.0	232
6	28/09/04	Parkfield, Eades	0.24	9.8	7.9	6.0	384
7	16/06/05	Redlands, SevenOaksDam	0.25	10	11.8	5.1	290
8	30/12/09	Holtville	0.26	42	6	5.8	322
9	09/08/07	Granada Hills, PorterRanch	0.27	6	7.5	4.6	148
10	18/05/09	Compton, Cressey Park	0.30	9	15.1	4.6	207
11	12/06/05	Mountain Center, PineMeadows R.	0.31	5	14.1	5.2	200
12	18/02/04	Cobb	0.32	2	3.6	4.4	213
13	31/10/07	San Jose, PrivateResidence	0.35	10	9.2	5.4	199
14	02/03/07	Martinez, VA Medical Clinic	0.39	10	16.6	4.4	149
15	22/12/03	San Luis Obispo, Rec. Center	0.40	61	7.6	6.4	162
16	04/04/10	CalexicoFireStation	0.40	62	10	7.2	266
17	07/07/10	Mountain Center, PineMeadows R.	0.75	20	11.7	5.4	185
18	28/06/92	Morongo Valley FireStation	0.81	28	5	6.5	198

19	28/02/01	Olympia, WDOTHighway Test Lab	0.82	18	59	6.8	250
20	10/01/10	Ferndale, LostCoastRanch	0.88	36	21.7	6.5	352

6. OBJECTIVE 1. EFFECT OF DAMPING

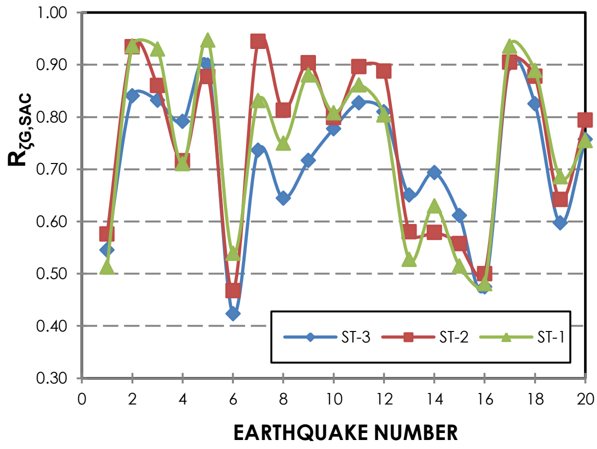
In order to estimate the individual effect of damping on the seismic response, elastic behavior of the structural models is considered. The different earthquake acceleration records are normalized with respect to the pseudo acceleration evaluated at the fundamental structural period ($S_a(T_1)$), in other words, for a given model, the earthquakes are scaled up or down in such a way that the ordinate values of their pseudo acceleration response spectra, evaluated at the fundamental period (T_1) of the model, is the same for all the records. The structural responses are estimated considering 0%, 2%, 5% and 10% of critical damping (ζ=0, 2, 5 and 10%), then the damping effect is estimated as

$$R_\zeta = \frac{Response\ (\zeta=2\%)}{Response\ (\zeta=0\%)}, \quad R_\zeta = \frac{Response\ (\zeta=5\%)}{Response\ (\zeta=2\%)} \quad \text{or} \quad R_\zeta = \frac{Response\ (\zeta=10\%)}{Response\ (\zeta=5\%)} \tag{3}$$

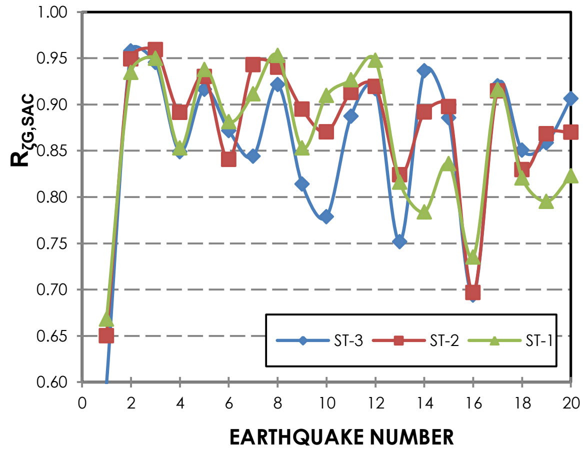
where, the damping reduction factor, R_ζ, represents the reduction of the response when damping is changed from 0 to 2%, from 2 to 5% or from 5 to 10%. These ranges of damping will be referred hereafter as *0-2*, *2-5* and *5-10* ranges, respectively. Additional subscripts are added to R_ζ to differentiate global from local response parameters or from one structural representation to another.

6.1 The SAC models

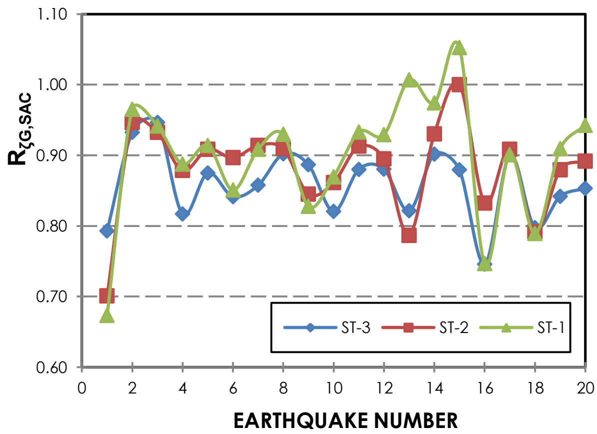
The symbol $R_{\zeta G,,SAC}$ is specifically used to represent the damping reduction factors for global response parameters of the SAC models. For a given model, earthquake, direction and interstory, the damping reduction factors for shears or displacements are estimated and averaged over all the plane frames for the interstory under consideration. Results for interstory shears are presented in Fig. 3 for Models SC1 and SC2 and the *N-S* direction. In this figure, the word "ST" stands for the story level. It can be observed that the $R_{\zeta G,SAC}$ values significantly vary from one earthquake to another, even thought the earthquakes were normalized with respect to $Sa(T_1)$. It reflects the effect of the earthquake frequency contents and the contribution of several modes on the structural responses. Values closer to 0.4 are observed in many cases for the *0-2* damping range indicating that increasing damping from 0 to 2% can reduce the response in almost 60%. It is also noted that the reduction in the response is, in general, larger for the *0-2* than for the *2-5* range which in turn is larger than that of the *5-10* range, confirming the well known results observed in typical SDOF systems: damping is more effective in reducing the response in low ranges. Results also indicate that the reduction in the response is larger for the upper interstory. Plots for $R_{\zeta G,,SAC}$ for the *E-W* direction were also developed but are not shown. The major conclusion made before apply to this case. The only additional observation that can be made is that the variation of $R_{\zeta G,SAC}$ from one story to another generally decreases as damping increases. The effect of damping on the reduction of the average interstory displacements is also estimated; considering two models, two directions and three cases of damping increments, as for the case of average interstory shears, 12 figures were developed, but they are not shown. A high correlation is observed between the plots of interstory shears and



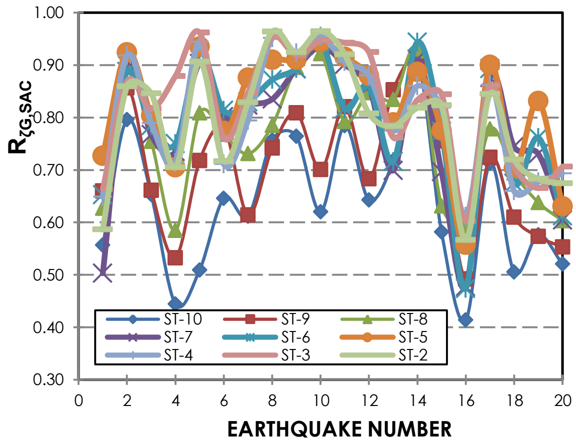
a) Model SC1, 0-2 range



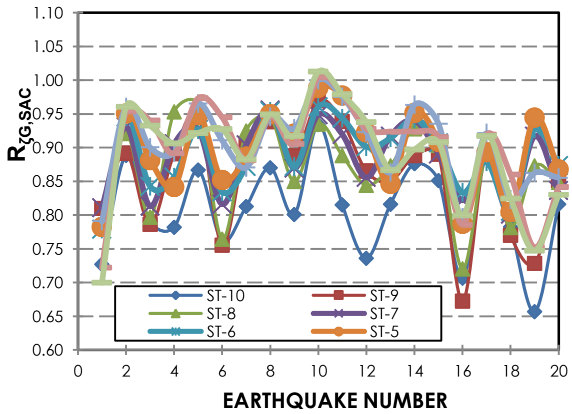
b) Model SC1, 2-5 range



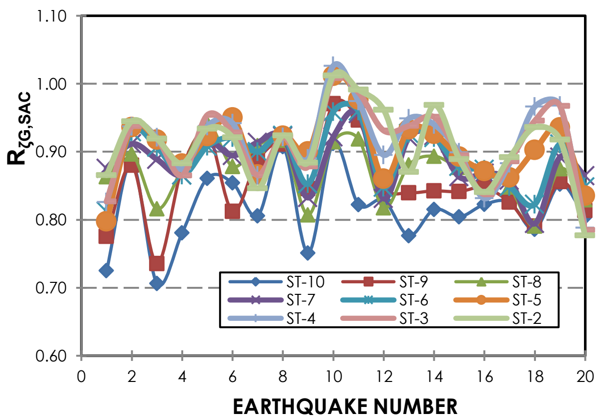
c) Model SC1, 5-10 range



d) Model SC2, 0-2 range

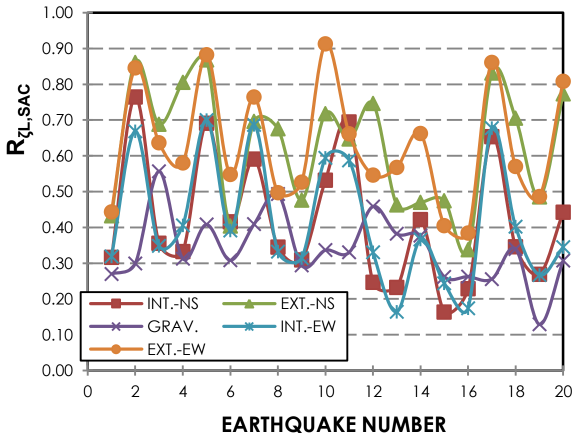


e) Model SC2, 2-5 range

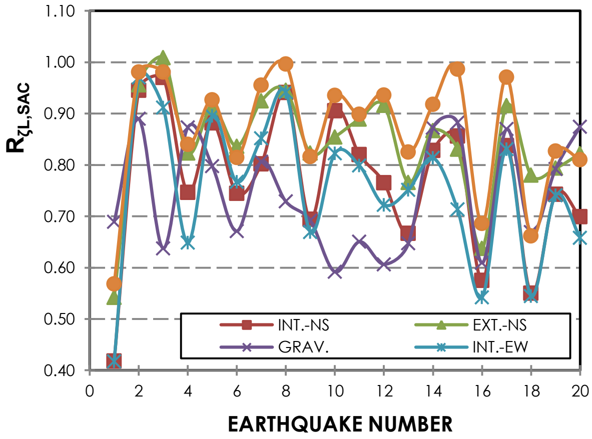


f) Model SC2, 5-10 range

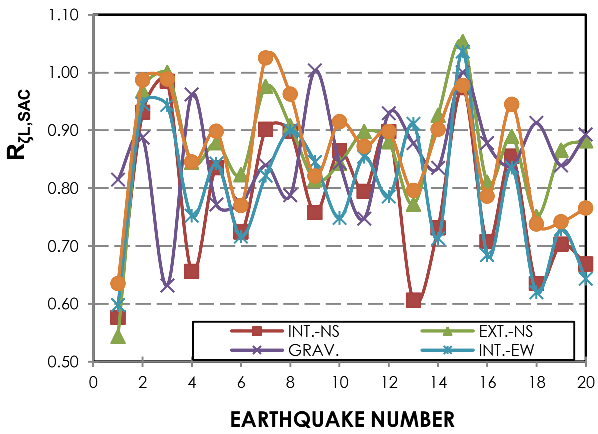
Figure 3. Global damping reduction factors for shear, SAC Models, N-S direction



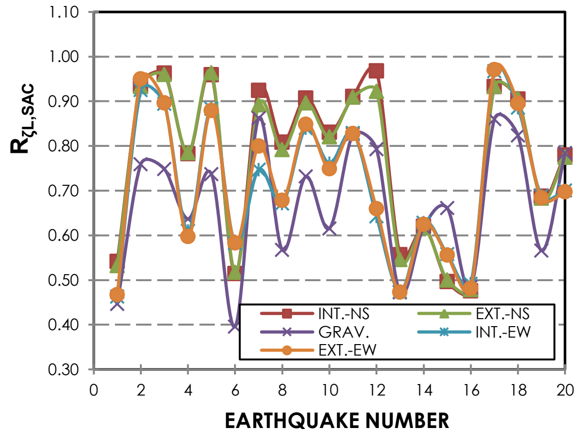
a) Axial load, 0-2 range



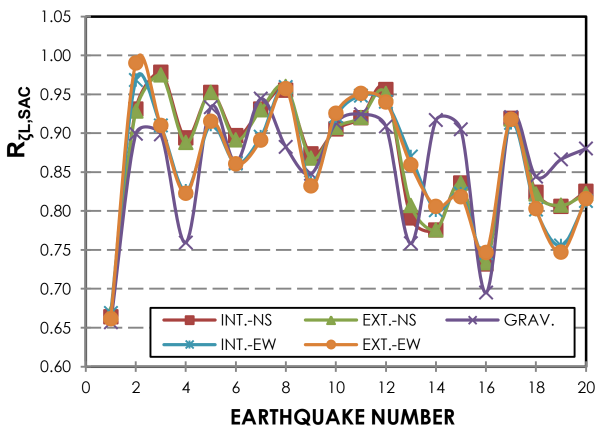
b) Axial load 2-5 range



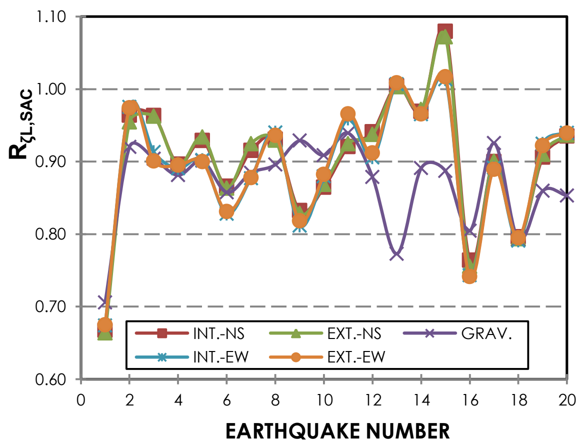
c) Axial load 5-10 range



d) Bending moment, 0-2 range



e) Bending moment, 2-5 range



f) Bending moment, 5-10 range

Figure 4. Local damping reduction factor for element forces, Model SC1

displacements. Thus, the major conclusions made before are valid for the displacement reduction.

The damping reduction factors for local response parameters ($R_{\zeta L,SAC}$) are considered next. Typical values of $R_{\zeta L,SAC}$ for axial loads and bending moments on selected members (Fig. 1c) of Model SC1 are given in Fig. 4. The results are similar in one sense to those of global response parameters but different in another: the $R_{\zeta L,SAC}$ values significantly vary from one earthquake to another and from one interstory to another; the damping reduction factors, however, seems to be smaller for local response parameters, particularly for axial loads, values lower than 0.20 are observed in some cases for the *0-2* damping range, implying a response reduction larger than 80%. The variation of $R_{\zeta L,SAC}$ from one column to another, as for $R_{\zeta G,SAC}$, generally decreases as damping increases and it is smaller for bending moment than for axial loads.

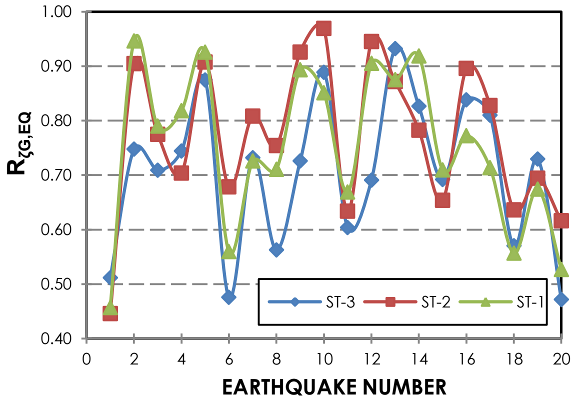
As commented above, most of the values of $R_{\zeta L,SAC}$ (Fig. 4) are smaller than unity implying that the response decreases as damping increases. For some cases, however, the values are slightly larger than unity implying that the response increases with an increment of damping, contradicting the results of typical SDOF system. The reason for this that the dynamic properties in terms of stiffness, natural frequencies, viscous damping, energy dissipation characteristics and the loading conditions of complex 3D systems, are quite different that those of typical SDOF systems and consequently their responses are expected to be different too.

6.2 The EQ Models

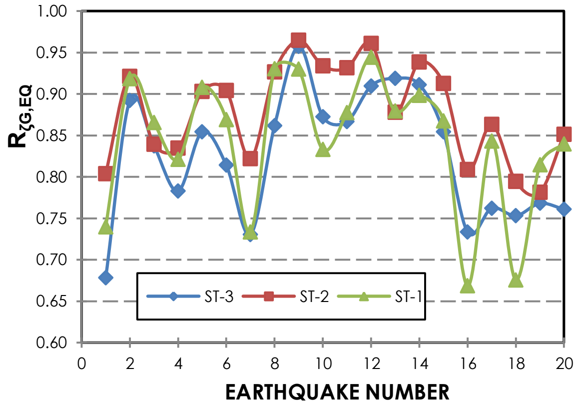
The $R_{\zeta G,EQ}$ parameter is used to represent the global damping reduction factors for the equivalent (EQ) 3D models. The results for interstory shears are presented in Fig. 5 for Models EQ1and EQ2 and the *N-S* direction. As for the 3D models with PMRF (SAC models), the reduction factors significantly vary from one earthquake to another and from one story to another reflecting the effect of earthquake frequency contents and the contribution of several modes of vibrations. From a comparison of all the plots, it is noted that the major observations made for the SAC models also apply the EQ Models, the only additional observation that can be made is that the reduction values are slightly larger for the SAC models. The local damping reduction factors for the EQ models ($R_{\zeta L,EQ}$) for both axial loads and bending moments are given in Fig. 6 for Model EQ1 and the N-S direction. The results resemble those of the SAC models in the sense that the reduction of the response is larger for local than for global parameters, larger for axial loads than for bending moments, and that the variation of the reduction factors from one column to another, which increases with damping, is smaller for bending moment than for axial load. For a given earthquake, the bending moment reductions, for the 2-5 or 5-10 ranges, are essentially the same for all the columns under consideration.

6.3 The 2D and SDOF Models

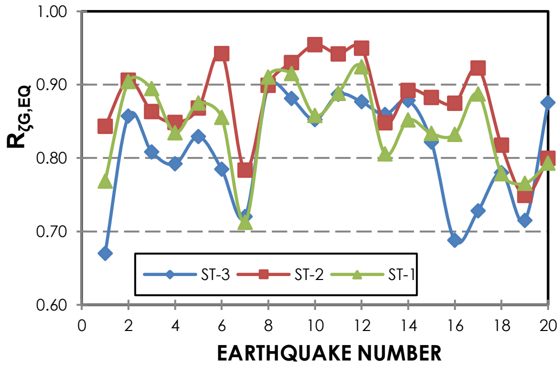
The global ($R_{\zeta G,2D}$) and local ($R_{\zeta L,2D}$) damping reduction factors, for shears and displacements, of the buildings modeled as plane structures , as well as those of the buildings modeled as SDOF systems ($R_{\zeta G,SDF}$ and $R_{\zeta l,SDF}$), are also calculated. Because of lack of space and because there are not significantly differences between these results and those of the SAC or the EQ models, the corresponding plots are not presented. It can be commented, however, that, in general, as for the SAC and EQ models, the reduction factors



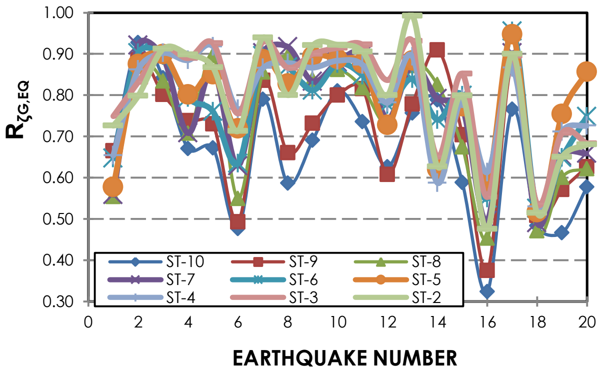
a) Model EQ1, 0-2 range



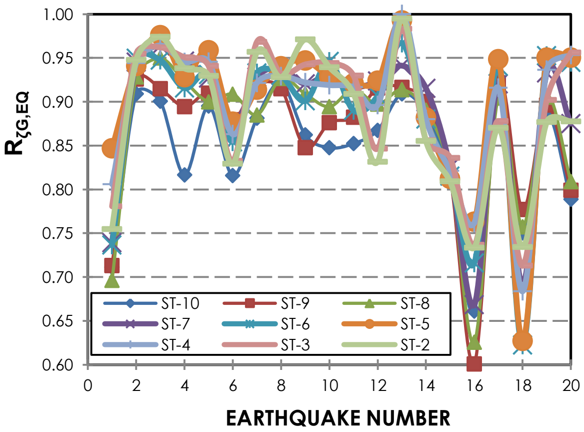
b) Model EQ1, 2-5 range



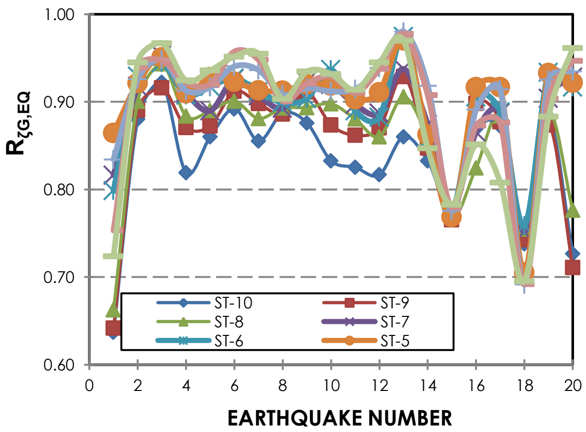
c) Model EQ1, 5-10 range



d) Model EQ2, 0-2 range



e) Model EQ2, 2-5 range



f) Model EQ2, 5-10 range

Figure 5. Global damping reduction factor for shear, EQ Models, N-S direction

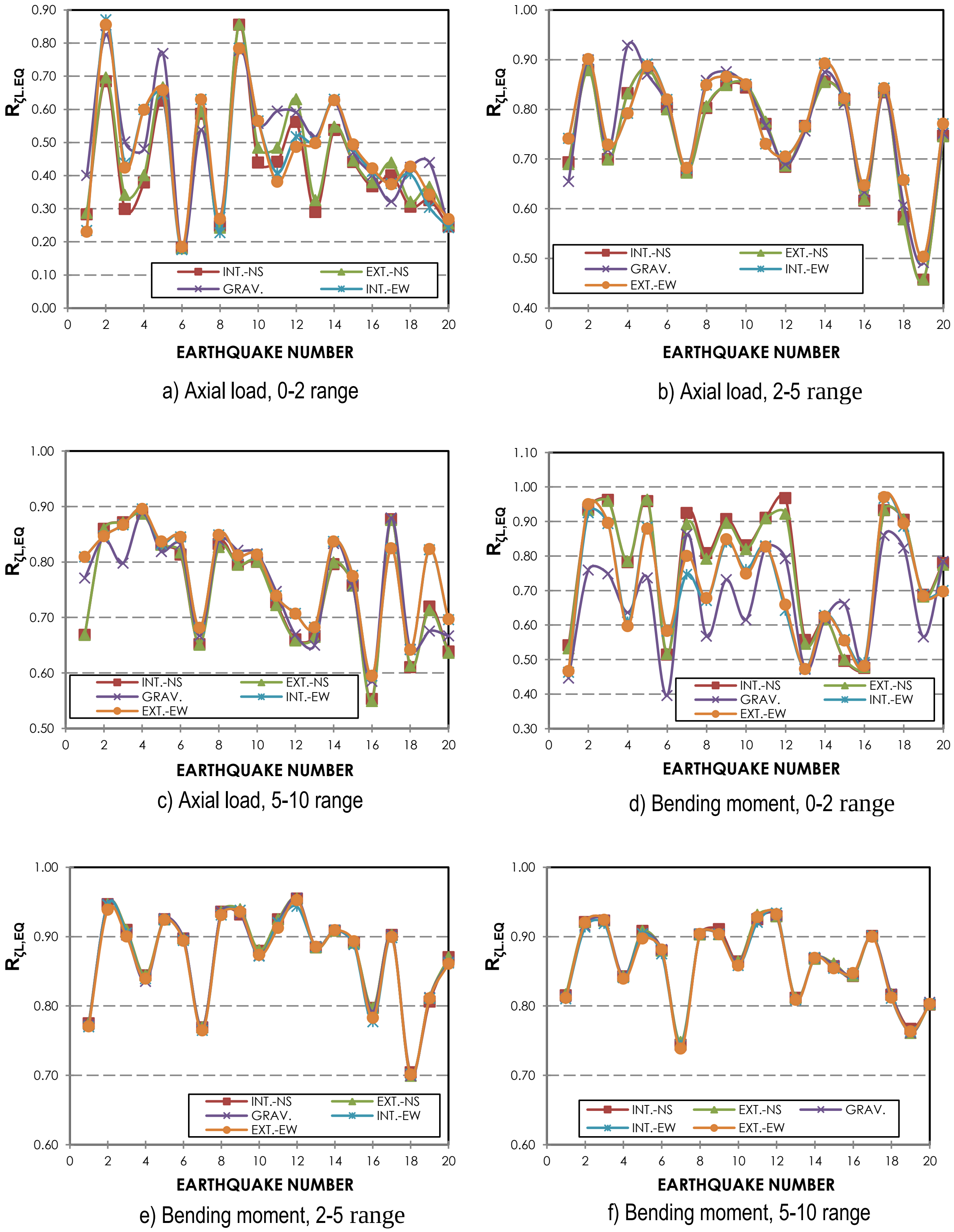


Figure 6. Local damping reduction factor for element forces, Model EQ1

Table 4. Statistics of damping global reduction factors ($R_{\zeta G}$) for shears and the 0-2 range.

EARTHQUAKE	$R_{\zeta G,SAC}$				$R_{\zeta G,EQ}$				$R_{\zeta G,2D}$				$R_{\zeta G,SDF}$			
	SAC1		SAC2		EQ1		EQ2		2D1		2D2		SD1		SD2	
	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
1	0.54	0.56	0.62	0.56	0.47	0.61	0.64	0.51	0.54	0.58	0.51	0.54	0.66	0.69	0.77	0.71
2	0.9	0.93	0.88	0.76	0.87	0.93	0.88	0.88	0.92	0.95	0.88	0.85	0.99	1.01	0.98	0.97
3	0.87	0.95	0.76	0.82	0.76	0.75	0.88	0.82	0.9	0.97	0.76	0.82	0.99	0.97	0.9	0.92
4	0.74	0.59	0.67	0.8	0.76	0.59	0.79	0.84	0.70	0.60	0.73	0.75	0.90	0.70	0.94	0.94
5	0.91	0.94	0.85	0.82	0.9	0.61	0.83	0.75	0.69	0.94	0.81	0.82	0.86	0.97	0.98	0.96
6	0.48	0.79	0.75	0.57	0.57	0.42	0.64	0.86	0.4	0.73	0.76	0.67	0.76	0.7	0.79	0.76
7	0.84	0.83	0.77	0.75	0.76	0.77	0.88	0.87	0.9	0.85	0.78	0.72	0.95	0.97	0.92	0.92
8	0.74	0.61	0.86	0.64	0.68	0.48	0.81	0.84	0.74	0.66	0.66	0.82	0.74	0.58	0.97	0.79
9	0.83	0.66	0.88	0.77	0.85	0.91	0.83	0.89	0.82	0.75	0.84	0.82	0.95	0.93	0.96	0.97
10	0.79	0.96	0.88	0.89	0.9	0.78	0.87	0.88	0.8	0.96	0.9	0.9	0.86	0.97	0.99	0.97
11	0.86	0.84	0.87	0.85	0.64	0.81	0.85	0.91	0.85	0.81	0.87	0.81	0.92	0.94	0.99	0.96
12	0.83	0.57	0.82	0.6	0.85	0.59	0.74	0.68	0.86	0.75	0.78	0.81	0.95	0.97	0.92	0.76
13	0.59	0.71	0.77	0.75	0.89	0.67	0.87	0.65	0.58	0.78	0.77	0.77	0.62	0.83	0.98	0.93
14	0.63	0.77	0.89	0.89	0.84	0.91	0.73	0.9	0.6	0.77	0.88	0.93	0.66	0.84	0.97	0.97
15	0.56	0.84	0.74	0.71	0.68	0.66	0.75	0.73	0.48	0.78	0.6	0.75	0.69	0.82	0.63	0.71
16	0.49	0.59	0.53	0.57	0.84	0.55	0.49	0.66	0.5	0.6	0.57	0.58	0.52	0.53	0.91	0.48
17	0.91	0.89	0.83	0.87	0.78	0.64	0.89	0.90	0.87	0.88	0.88	0.88	0.93	0.97	0.94	0.94
18	0.86	0.62	0.67	0.62	0.59	0.63	0.51	0.56	0.64	0.50	0.64	0.66	0.58	0.5	0.9	0.99
19	0.64	0.51	0.68	0.68	0.7	0.68	0.64	0.61	0.5	0.67	0.69	0.6	0.81	0.48	0.94	0.81
20	0.77	0.91	0.62	0.85	0.54	0.75	0.69	0.67	0.73	0.67	0.56	0.54	0.51	0.87	0.71	0.6
MEAN	0.74	0.75	0.77	0.74	0.74	0.69	0.76	0.77	0.70	0.76	0.74	0.75	0.79	0.81	0.91	0.85
COV	0.20	0.20	0.14	0.15	0.17	0.20	0.16	0.16	0.23	0.18	0.16	0.16	0.20	0.22	0.11	0.17

Table 5. Statistics of local damping reduction factors ($R_{\zeta L}$) for the 0-2 range

PARAMETER			$R_{\zeta L,SAC}$				$R_{\zeta L,EQ}$				$R_{\zeta L,2D}$				$R_{\zeta L,SDF}$			
			SAC1		SAC2		EQ1		EQ2		2D1		2D2		SD1		SD2	
			N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
A X I A L	E X T	MEAN	0.63	0.63	0.62	0.53	0.45	0.48	0.55	0.58	0.44	0.53	0.57	0.69	0.44	0.48	0.59	0.62
		COV	0.27	0.26	0.24	0.29	0.39	0.38	0.32	0.33	0.30	0.25	0.34	0.22	0.44	0.35	0.34	0.32
	I N T	MEAN	0.42	0.42	0.50	0.49	0.43	0.47	0.52	0.47	0.35	0.37	0.40	0.46	0.42	0.46	0.49	0.54
		COV	0.43	0.42	0.35	0.35	0.40	0.40	0.32	0.36	0.33	0.41	0.41	0.38	0.42	0.44	0.33	0.38
M O M E N T	E X T	MEAN	0.77	0.72	0.81	0.80	0.77	0.72	0.81	0.80	0.44	0.75	0.44	0.80	0.79	0.79	0.90	0.90
		COV	0.23	0.23	0.14	0.14	0.23	0.23	0.14	0.14	0.30	0.21	0.30	0.17	0.20	0.20	0.11	0.11
	I N T	MEAN	0.77	0.71	0.81	0.80	0.77	0.71	0.81	0.80	0.35	0.76	0.35	0.80	0.79	0.79	0.91	0.91
		COV	0.23	0.22	0.14	0.15	0.23	0.22	0.14	0.15	0.33	0.22	0.33	0.18	0.20	0.20	0.11	0.11

are larger for global than for local parameters, particularly for the case of axial load, and that the variation of the reduction factors from one structural element to another is larger for axial loads than for bending moments.

6.4 Results in terms of statistics

The global reduction factors of the SAC, EQ and 2D models are averaged over all the stories and then their statistics are estimated over all the earthquakes. For the case of the SDF models their statistics are estimated over all the earthquakes. The results for shears are given in Table 4 for the 0-2 range respectively. The statistics for the 0-2 and 5-10 ranges are not presented but some comments are made. It is observed that for the

0-2 range, the mean values of the reduction factors range from 0.70 to 0.91. The largest and smallest values are observed for the SDF and the 2D models, respectively, and they are quite similar for the SAC and the EQ models. The uncertainty in the estimation is moderate, the coefficient of variation ranges from 0.11 to 0.22. For the 2-5 range, the mean reduction factors are quite similar for the four structural representations, which in turn are larger (implying a smaller shear reduction) than those of the 0-2 range. The uncertainty in the estimation is, however, much smaller for the 2-5 than for the 0-2 range. The statistics for displacements are not presented. However, from a comparison of the mean values of the reduction factors for shear and displacements, it is observed that the mean and the COV values are quite similar, indicating a high correlation between these two parameters.

The statistics for local response parameters are given in Table 5 for the 0-2 range. The variation of the mean reduction factors for the 0-2 range from one structural representation to another, from one model to another, from one response parameter to another or from one direction to another is larger for local than for global response parameters The minimum observed value (greater response reduction) is observed to be 0.35 for axial load at interior columns of the N-S direction of the 3-level plane model while the largest one (minimum response reduction) is 0.91 for bending moment at interior column of the SDOF model of the 10-level building. The most important observation that can be made is that the reduction factors can be significantly smaller for local than for global response parameters, as concluded before from particular figures. On the other hand, the uncertainty in the estimation of the reduction factors may be significantly larger for local response parameters, particularly for the case of the 0-2 range.

7. OBJECTIVE 2. EFFECT OF YIELDING

The effect of yielding on the seismic response is discussed in this section of the paper. In order to estimate the reduction in the response produced only by yielding of the material, for a given amount of damping, the elastic and inelastic responses are compared. 2%, 5% and 10% of critical damping are considered. The different earthquake acceleration records are first normalized with respect to the pseudo acceleration evaluated at the fundamental structural period ($S_a(T_1)$) and then, they are uniformly scaled up in such a way that considerable yielding occurs in any of the models for the critical earthquake. The maximum interstory displacement developed was about 1.8% for the 2D models; they were smaller for the SAC and EQ models than for the 2D models. It was observed that about 6 to 23 plastic hinges were formed in the cases where yielding occurred. The yielding reduction factor is estimated as:

$$R_P = \frac{Inelastic\ response\ (\zeta=2\%)}{Elastic\ response\ (\zeta=2\%)},\ R_P = \frac{Inelastic\ response\ (\zeta=5\%)}{Elastic\ response\ (\zeta=5\%)}\ \text{or}\ R_P = \frac{Inelastic\ response\ (\zeta=10\%)}{Elastic\ response\ (\zeta=10\%)} \quad \textbf{(4)}$$

Additional subscripts are added to R_P to differentiate global from local response parameters or from one structural representation to another. As for the case of damping reduction factors, 16 figures were developed for each structural representation. In spite of there are some plots that deserve to be particularly discussed, only the statistics for interstory shears will be presented because of lack of space. The statistics for the global yielding reduction factors (R_{PG}) as well as the average results for individual earthquakes, are first discussed. They are given in 6 for ζ=2% . It is observed that, for a given structural representation, the reduction factors can significantly vary from one earthquake to another without showing any trend; they vary from 0.75 to 1.04, from 0.65 to 1.06, and from 0.50 to 0.86 for the SAC, EQ and 2D models, respectively. For the case of equivalent SDF models the yielding was not significant, the reduction factors resulted to be close to unity practically in all cases. From the individual and the mean values of R_{PG} it is observed that the reduction is about 20% larger for the 2D models than for the SAC or the EQ models. The uncertainty in the estimation is small in all the cases and it is slightly larger for 5% than for 2% damping.

Table 6. Statistics of global yielding reduction factors (R_{PG}) for shears and ζ=2%.

EARTHQUAKE	$R_{PG,SAC}$				$R_{PG,EQ}$				$R_{PG,2D}$				$R_{PG,SDF}$			
	SAC1		SAC2		EQ1		EQ2		2D1		2D2		SD1		SD2	
	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
1	0.97	0.97	0.89	0.93	0.99	0.97	0.98	0.91	0.72	0.66	0.50	0.72	1.00	1.00	1.01	1.03
2	0.97	0.87	0.75	0.85	0.87	0.81	0.72	0.82	0.71	0.72	0.67	0.79	0.95	1.00	1.00	1.01
3	0.94	1.03	0.82	0.75	0.94	0.95	0.86	0.95	0.91	0.79	0.66	0.72	1.00	0.97	1.01	1.01
4	0.98	0.84	0.94	0.87	0.99	1.03	0.97	0.90	0.61	0.63	0.72	0.79	1.00	1.00	1.01	0.97
5	0.93	0.74	0.79	0.83	0.79	0.74	0.78	0.98	0.77	0.73	0.78	0.83	1.00	1.00	1.01	1.01
6	0.94	0.96	0.90	0.81	0.94	0.98	0.84	0.85	0.77	0.77	0.66	0.69	1.00	1.00	1.01	1.01
7	0.96	1.07	0.92	0.90	0.98	0.92	0.76	0.80	0.82	0.86	0.67	0.71	1.00	1.00	1.00	1.00
8	0.91	1.11	0.76	0.80	0.92	0.83	0.75	0.78	0.73	0.78	0.75	0.76	1.00	1.00	1.00	1.01
9	0.92	0.99	0.86	0.83	1.00	0.88	0.95	0.82	0.74	0.62	0.80	0.77	1.00	1.00	1.00	1.01
10	0.83	0.71	1.15	0.85	0.97	0.76	0.72	0.76	0.80	0.70	0.79	0.79	1.00	1.00	0.99	1.00
11	0.77	0.75	0.90	0.83	0.96	0.95	0.75	0.65	0.72	0.80	0.76	0.83	1.00	1.00	1.00	1.01
12	1.02	0.81	0.98	0.92	1.01	0.84	0.74	0.90	0.72	0.76	0.70	0.79	1.00	1.00	1.02	1.01
13	1.00	0.83	0.88	0.82	0.88	0.91	1.06	0.94	0.75	0.80	0.66	0.81	0.94	1.00	1.00	1.00
14	0.81	0.70	0.73	0.78	0.84	0.79	0.73	0.74	0.74	0.71	0.70	0.76	1.00	1.00	1.01	1.02
15	0.96	0.96	0.78	0.90	0.74	1.03	0.79	0.91	0.73	0.85	0.69	0.79	1.00	1.00	1.00	1.00
16	0.97	0.99	0.98	0.84	1.00	0.91	1.00	0.98	0.64	0.59	0.72	0.74	1.00	1.00	1.01	1.00
17	0.74	0.87	0.75	0.84	0.83	1.06	0.93	0.81	0.78	0.76	0.72	0.82	1.00	0.96	1.01	1.01
18	0.95	0.98	0.98	0.96	0.99	1.00	0.98	0.98	0.69	0.56	0.71	0.62	0.97	1.00	1.03	0.98
19	0.96	0.88	0.75	0.95	0.83	0.96	1.26	0.97	0.65	0.67	0.66	0.75	1.00	1.00	1.01	1.00
20	1.05	0.75	0.96	1.08	0.84	1.00	0.87	1.01	0.70	0.65	0.62	0.70	1.00	1.00	1.01	1.01
MEAN	0.93	0.89	0.87	0.87	0.92	0.92	0.88	0.87	0.74	0.72	0.70	0.76	1.00	1.00	1.01	1.01
COV	0.09	0.14	0.12	0.09	0.09	0.11	0.18	0.11	0.09	0.12	0.10	0.07	0.02	0.01	0.01	0.01

The statistics for the local yielding reduction factors are given in Table 7 for ζ=2%. It is observed that, like the yielding reduction factors of global response parameters, they can significantly vary from one structural representation to another and from one response parameter to another. The values resulted smaller for axial loads than for bending moments and smaller for the EQ than for the others structural representations. The minimum observed reduction factors were for the axial loads of the EQ models, which range from 0.17 to 0.25 for the case of 5% damping while the maximum observed reduction factors were for bending moments of the SAC models which range from 0.92 to 1.19 for 5% damping. The uncertainty in the estimation is larger for local than for global yielding reduction factors .

Table 7. Statistics of yielding local reduction factors (**R_{PL}**) for ζ=2%

PARAMETER			$R_{PL,SAC}$				$R_{PL,EQ}$				$R_{PL,2D}$				$R_{PL,SDF}$			
			SAC1		SAC2		EQ1		EQ2		2D1		2D2		SD1		SD2	
			N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
AXIAL	EXT	MEAN	0.60	0.55	0.96	0.87	0.32	0.30	0.22	0.23	0.65	0.57	0.53	0.56	0.83	0.71	0.40	0.38
		COV	0.17	0.18	0.31	0.29	0.48	0.63	0.37	0.36	0.20	0.26	0.14	0.13	0.23	0.23	0.36	0.25
	INT	MEAN	0.32	0.25	0.24	0.20	0.31	0.30	0.21	0.26	0.97	1.08	0.73	0.96	1.01	0.80	0.63	0.46
		COV	0.44	0.36	0.30	0.30	0.48	0.63	0.45	0.40	0.07	0.22	0.34	0.20	0.26	0.25	0.32	0.28
MOMENT	EXT	MEAN	0.97	1.10	0.91	1.06	0.86	0.85	0.87	1.00	0.69	0.66	0.67	0.79	1.00	1.00	1.01	1.01
		COV	0.13	0.13	0.07	0.15	0.17	0.18	0.21	0.12	0.10	0.11	0.11	0.07	0.01	0.00	0.01	0.01
	INT	MEAN	0.97	1.10	0.90	1.06	0.85	0.87	0.89	0.98	0.74	0.74	0.77	0.87	0.98	0.99	1.01	1.01
		COV	0.13	0.13	0.07	0.15	0.17	0.19	0.21	0.14	0.18	0.20	0.13	0.06	0.01	0.01	0.01	0.01

From a comparison of the damping and yielding global reduction factors, it is observed that, excepting those of the 2D models, they are smaller for damping implying a larger reduction in the global structural response. The uncertainty in the estimation is, excepting that of the SDF models, larger for the case of yielding. For the case of local reduction factors, they are smaller for yielding for axial loads; for bending moments, however, they resulted to be smaller for damping. For both cases the uncertainty in the estimation can be considerable.

As stated earlier, in the estimation of both, damping and yielding reduction factors, in order to have the same participation of the fundamental structural mode, the different earthquake acceleration records were first normalized with respect to the pseudo acceleration evaluated at the fundamental structural period ($S_a(T_1)$) and then they were uniformly scaled up in such a way that for the critical earthquake considerable yielding occurred in any of the structural representations. The maximum interstory displacement developed was about 1.8% for the 2D models which corresponds to a deformation state close to the collapse. However, yielding was not significant for many of the earthquakes even for the case of 2D models and consequently the yielding reduction factors don't represent the maximum ones that could have been developed in the models. Thus, the conclusion made in relation with the yielding of the material are for the particular level of structural deformation and could significantly change if more yielding is allowed in the structures. For those cases where considerable yielding occurred in the models (as for the 2D models), the global o local reduction factors are comparable or even much larger for the case of damping.

CONCLUSIONS

The evaluation of the effect of viscous damping and yielding of the material on the reduction of the seismic responses of steel buildings with perimeter moment resisting frames, modeled as three-dimensional (3D) complex multi-degree of freedom (MDOF) systems, constitutes the main objective of this paper. The results are compared with those of equivalent 3D structural representations with spatial moment resisting frames as well as with those of bi-dimensional and equivalent single degree of freedom idealizations. Two steel model buildings subjected to twenty recorded strong motions scaled in terms of the spectral acceleration at the fundamental mode of vibration of the structures ($S_a (T_1)$), are used in the study. The effects are expressed in terms of global and local damping reduction factors and in terms of global and local yielding reduction factors.

The results indicate that the magnitude of the response reduction significantly vary from one earthquake to another, even thought the earthquakes were normalized with respect to the same pseudo acceleration, reflecting the influence of the earthquake frequency contents and the contribution of several vibration modes on the structural responses. It is also observed that the reduction in the response produced by damping may be larger or smaller than that of yielding. This reduction can significantly vary from one structural representation to another and is smaller for global than for local response parameters, which in turn depends on the particular local response parameter and the location of the structural element under consideration. The reason for this is that the dynamic properties in terms of stiffness, natural frequencies, viscous damping, energy dissipation characteristics as well as loading conditions of complex 3D systems, are quite different than those of simplified bi-dimensional or SDOF idealizations, consequently their responses are expected to be different too. The uncertainty in the estimation is significantly larger for local than for global response parameters and decreases as damping increases. It is also noted that the reduction in the response is, in general, larger for low ranges of damping confirming what observed from the results of typical SDOF systems. Based on the results of this study, it is concluded that, estimating the effect of damping and yielding on the seismic response of steel buildings by using simplified models may be a very crude approximation. Moreover, because of the significant differences between the damping and yielding reductions, the effect of yielding should explicitly calculated by using complex 3D MDOF models instead of estimating it in terms of equivalent viscous damping.

ACKNOWLEDGEMENTS

This paper is based on work supported by La Universidad Autónoma de Sinaloa (UAS) under grant PROFAPI-2012/148. 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.

REFERENCES

Ayoub, A. and Chenouda, M. (2009), “Response spectra of degrading structural systems”, *Engineering Structures,* **31**, 1393-1402.

Arroyo-Espinoza, D. and Teran-Gilmore, A. (2003), “Strength reduction factors for ductile structures with passive energy dissipating devices”, *Journal of Earthquake Engineering*, **7**(2) 297–325.

Bathe KJ. Finite Element Procedures in Engineering Analysis 1982. Prentice-Hall, Englewood Cliffs, New Jersey.

BOCA (1993), 12th Edition Building Officials & Code Administration International Inc, *National Building Code*

Bojorquez, E., Reyes-Salazar A., Terán-Gilmore A. and Ruiz, S.E. (2010), "Energy-based damage index for steel structures", *Steel and Composite Structures An International Journal*, (**10**) 4, 331-348.

Chopra, A.K. (2007), *Dynamics of Structures*, Prentice Hall, New Jersey.

Ceylan, M., Arslan, M.H., Ceylan, R., Kaltakci, M.Y. and Ozbay, Y. (2010), "A new application area of ANN and ANFIS: determination of earthquake load reduction factor of prefabricated industrial buildings", *Civil Engineering and Environmental Systems,* **27**(1) 53-69.

Cai, J., Zhou, J. and Fang X. (2006), "Seismic Ductility Reduction Factors for Multi-Degree-of-Freedom Systems", *Advances in Structural Engineering,* **9**(5) 591-601.

Clough RW, Penzien J. Dynamic of Structures, 2nd edition. McGraw Hill, New York, 1993

Federal Emergency Management Agency (2000), State of the Art Report on Systems Performance of Steel Moment Frames Subjected to Earthquake Ground Shaking, SAC Steel Project, Report FEMA 355C.

Gao, L. and Haldar, A. (1995), "Nonlinear Seismic Analysis of Space Structures with PR connections", *International Journal of Microcomputers in Civil Engineering,* **10**, 27-37.

Ganjavi, B. and Hao, H. (2012), "Effect of structural characteristics distribution of strength demand and ductility reduction factor of MDOF systems considering soil-structure interaction", *Earthquake Engineering and Engineering Vibration",* **11**, 205-220.

Hong, H.P. and Jiang, J. (2004), "Ratio between Peak inelastic and elastic responses with uncertain structural properties", *Canadian Journal of Civil Engineering,* **31**(4) 703-711.

Hadjian, A.H. (1989), "An Evaluation of the Ductility Reduction Factor Q in the 1976 Regulations for the Federal District of Mexico", *Earthquake Engineering and Structural Dynamics*, **18**, 217-231.

Karmakar D. and Gupta V.K. (2006), "A parametric study of strength reduction factors for elasto-plastic oscillators", Sādhanā, **31**(4) 343-357.

Kondoh K, Atluri SN. Large deformation, elasto-plastic analysis of frames under non-conservative loading using explicitly derived tangent stiffness based on assumed stress. Comp. Mech.,1987; 2(1):1-25.

Levy, R., Rutenberg, A. and Qadi, Kh. (2006), "Equivalent linearization applied to earthquake excitations and the R-μ-T_0relationships", *Engineering Structures,* **28**(2) 216-228.

Miranda, E. and Bertero V. (1994), "Evaluation of Strength Reduction Factors for Earthquake-Resistant Desing*", Earthquake Spectra*, **10**(2), 357-379

Mahadevan S, Haldar A. Stochastic FEM-based evaluation of LRFD. Journal of the Structural Engineering Division ASCE 1991;117(5):1393-1412.

Nassar, A. and Krawinkler, H. (1991), "Seismic Demands of SDOF and MDOF Systems, *John A. Blume Earthquake Engineering Center,* Report No. 95, Stanford University.

Newmark, N.M. and Hall, W.J. (1982), *Earthquake Spectra and Design*, Monograph Series. Berkeley, California, Earthquake Engineering Research Institute.

Reyes-Salazar, A. and Haldar, A. (1999), "Nonlinear Seismic Response of Steel Structures with Semi-rigid and Composite Connections", *Journal of Constructional Steel Research*, 51, 37-59.

Reyes-Salazar, A. and Haldar, A. (2000), “Dissipation of Energy in Steel Frames With PR Connections”, *Structural Engineering and Mechanics, an International Journal,* **9**(3), 241-256.

Reyes-Salazar, A. and Haldar, A. (2001a) “Energy Dissipation at PR Frames Under Seismic Loading”, *Journal of Structural Engineering ASCE*, **127**(5), 588-593.

Reyes-Salazar, A. and Haldar, A. (2001b), “Seismic Response and Energy Dissipation in Partially Restrained and Fully Restrained Steel Frames: An Analytical Study”, *Steel & Composite Structures, An International Journal,* **1**(4), 459-480.

Reyes-Salazar, A. (1997), “Inelastic Seismic Response and Ductility Evaluation of Steel Frames with Fully, Partially Restrained and Composite Connections”, *PhD. Thesis, Department of Civil Engineering and Engineering Mechanics*, University of Arizona, Tucson, AZ.

Reyes-Salazar, A. (2002), “Ductility and Ductility Reduction Factors”, *Structural Engineering and Mechanics”, International Journal,* **13**(4), 369-385.

Sanchez-Ricart, L. (2010), “Assessment and management of Risk for Engineered Systems and Geohazards”, *Georisk*, **4**(4)**,** 208-229.

Shen J. and Akbas B. (1999), “Seismic energy demand in steel moment frames”, Journal of Earthquake Engineering, 3(4), 519-559.

Uniform Building Code (1997), Int. Conf. of Building Officials (ICBO), Whittier, California.