EVALUATION OF STRUCTURAL RELIABILITY OF STEEL FRAMES CONSIDERING CUMULATIVE DAMAGE

E. Bojórquez[1], A. Terán-Gilmore[2], S. E. Ruiz[3] and A. Reyes-Salazar[4]

[1] *PhD, Dipartimento di Ingegneria Strutturale, Università di Napoli Federico II, Naples Italy*
[2] *Professor, Departamento de Materiales, Universidad Autónoma Metropolitana, México City*
[3] *Professor, Instituto de Ingeniería ,Universidad Nacional Autónoma de México, México City*
[4] *Professor, Facultad de Ingeniería, Universidad Autónoma de Sinaloa, Culiacán, Sinaloa, México*
Email:bojorquezeden.mora@unina.it, sruizg@iingen.unam.mx, tga@correo.azc.uam.mx, reyes@uas.uasnet.mx

ABSTRACT:

The structural reliability of mid-rise steel buildings located in the Lake Zone of Mexico City is evaluated. Two earthquake demand parameters are used for this purpose: a) Maximum interstory drift; and b) Normalized plastic hysteretic energy. The buildings, designed according to the Mexico City Building Code (RCDF-2004), have a structural system constituted by regular moment-resistant steel frames. The demand hazard curves of the buildings are expressed in terms of both parameters. The curves are compared and used to provide a general idea of the reliability levels associated to the set of buildings. Large differences are observed in the structural reliabilities derived from the maximum interstory drift and the normalized plastic hysteretic energy. In particular, the reliability levels obtained through the use of normalized plastic hysteretic energy tend to decrease considerably with respect to those derived from maximum interstory drift at state of structural failure.

KEYWORDS: Cumulative damage, maximum interstory drift, normalized plastic hysteretic energy, structural reliability

1. INTRODUCTION

The maximum interstory drift and/or the maximum ductility demand are response parameters targeted by current seismic design codes. Nevertheless, in some cases damage caused by earthquakes can not be explained exclusively through maximum deformation demands. Particularly, the seismic performance of structures located on soft soils or subjected to long duration ground motions can be strongly influenced by the structural damage and deterioration accumulated through several cycles of plastic deformation (Rodríguez and Ariztizabal 1999, Bojórquez and Ruiz 2004, Arroyo and Ordaz 2007, Teran and Jirsa 2007). Several studies suggest that the plastic hysteretic energy normalized by the force and displacement at first yield can be used to assess structural damage suffered by structures subjected to severe plastic cycling (Darwin and Nmai, 1985; Teran and Baena, 2008). Due to the importance of normalized plastic hysteretic energy to represent the structural damage, it is necessary to evaluate the structural reliability in terms of this parameter. For this reason, in this paper the structural reliability expressed in terms of two different response parameters, of mid-rise steel buildings designed according to the Mexico City Building Code is compared. The first parameter used for this purpose is the maximum interstory drift, commonly used by most of current seismic codes as the target parameter to promote an adequate performance of earthquake-resisting structures. Because of the close relation with the structural damage exhibited by structures subjected to severe plastic cycling, the second parameter under consideration is the normalized plastic hysteretic energy. It will be shown that the reliability levels obtained through the consideration of the plastic hysteretic energy can be smaller than those derived from maximum interstory drift.

2. STRUCTURAL MODELS AND EARTHQUAKE MOTIONS

2.1 Structural Models

Four mid-rise moment-resistant steel frames were considered. The frames were designed according to the Mexico City Building Code. All the frames have three bays with a length of 8m. The number of floors is 4, 6, 8 and 10; and the height of all stories is 3.5m. Each frame was designed for ductile detailing and a seismic behavior factor Q equal to 3. A36 steel was used for the structural elements. A bilinear hysteretic model with 3% strain-hardening was used to model the cyclic behavior of the steel elements. 3% of critical damping was used for the first two periods of the frames through a Rayleigh matrix. Relevant characteristics, such as the fundamental period of vibration (T_1), and the seismic coefficient and displacement at first yield (C_y and D_y, respectively) for all the frames are shown in Table 1 (C_y and D_y were obtained from push-over analyses).

Table 1. Dynamic characteristics of the steel frames

Frame	T_1 (s)	C_y	D_y (m)
F4	0.90	0.45	0.136
F6	1.07	0.42	0.174
F8	1.20	0.38	0.192
F10	1.37	0.36	0.226

2.2 Ground Motion Records

A set of 23 narrow-banded ground motions recorded at soft-soil sites located in the valley of Mexico was used. All motions within the set exhibit a dominant period of vibration close to 2s. Figure 1 illustrates, for all records in the set, the elastic response spectra corresponding to 3% of critical damping. As shown, the narrow-banded records were selected in such way that they exhibit similar spectral shapes. This was done so that a scaling criterion based on the spectral acceleration at the first mode of vibration could be used.

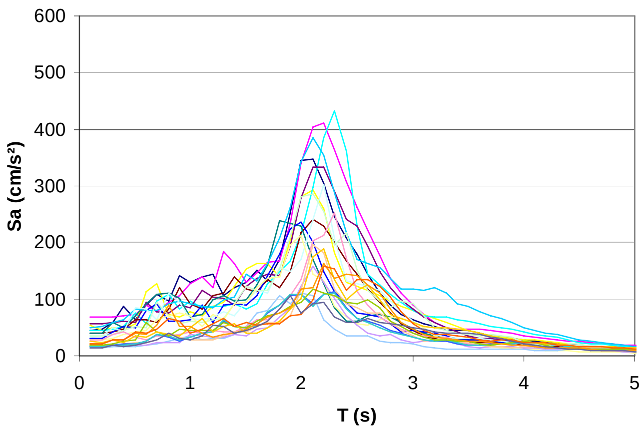

Figure 1 Elastic response spectra for all records under consideration, 3% of critical damping

3. EVALUATION OF STRUCTURAL RELIABILITY

One of the main objectives of Earthquake Engineering is to quantify, through the consideration of all possible earthquake ground motion intensities at a site, the seismic reliability implicit in structures. Probabilistic seismic demand analysis (PSDA) is used as a tool for estimating the reliability of structures through the evaluation of the mean annual frequency of exceeding a specified value of an earthquake demand parameter *EDP* (e.g. interstorey drift, normalized plastic hysteretic energy). Based on past studies (Esteva, 1967; Cornell, 1968) and considering the total probability theorem, the probabilistic seismic demand analysis can be carried out through the consideration of the mean annual rate of exceeding a given value of *EDP*:

$$\lambda_{EDP}(x) = \int_{IM}\int_{M}\int_{R} P[EDP > x \mid IM, M, R] f(IM \mid M, R) f(M, R)\, dr\, dm\, d(im) \tag{3.1}$$

where $\lambda_{EDP}(x)$ is the mean annual frequency of *EDP* exceeding a value of *x*, $f(IM \mid M, R)$ is the conditional distribution function of the intensity measure (*IM*) given values of magnitude (*M*) and distance (*R*), $f(M, R)$ is the joint probability density function of *M* and *R*, and finally, $P[EDP > x \mid IM, M, R]$ is the failure probability of the structure as a function of *IM*, *M* and *R* (fragility curves, see next subsection). If $P[EDP > x \mid IM, M, R] = P[EDP > x \mid IM]$, then the *IM* is said to be *sufficient* (Shome, 1999) since its ability to predict the structural response is independent of *M* and *R*, given *IM*. It has been shown that the spectral acceleration at first mode of vibration $Sa(T_1)$ is *sufficient* with respect to magnitude and distance (Shome, 1999). However, it is important to point out that under some circumstances $Sa(T_1)$ is not a good predictor of the nonlinear structural response, and more appropriate *IM* measures are necessary. For example, the vector $<Sa, \varepsilon>$, which is related to the elastic spectral shape, has resulted sufficient and efficient in many cases (Baker and Cornell, 2005). Other such measures include the advanced scalar *IM* proposed by Tothong and Luco (2007); the vector *IM* $<Sa, R_{T1,T2}>$ proposed by Baker and Cornell (2007) ($R_{T1,T2}$ is the ratio between the spectral acceleration at period T_2 divided by spectral acceleration at period T_1); and the vector $<Sa, N_p>$ proposed by (Bojórquez et al, 2008), which is related to the spectral shape. As mentioned before, the records used herein were selected in such a manner that a scaling criteria based on $Sa(T_1)$ could be used. Due to the sufficiency of $Sa(T_1)$ with respect to *M* and *R*, Eqn. 3.1 can be expressed as:

$$\lambda_{EDP}(x) = \int_{Sa(T_1)} P[EDP > x \mid Sa(T_1) = sa]\, d\lambda_{Sa(T_1)}(sa) \tag{3.2}$$

where $d\lambda_{Sa(T_1)}(sa) = \lambda_{Sa(T_1)}(sa) - \lambda_{Sa(T_1)}(sa + dsa)$ is the differential of the ground motion hazard curve expressed in terms of $Sa(T_1)$. Eqn. 3.2 was used to evaluate the structural reliability in terms of two *EDPs*, for the steel frames subjected to the narrow-band motions.

3.1 Fragility curves

The fragility curves were obtained through the consideration of a lognormal distribution. The probability that *EDP* exceeds *x* given $Sa(T_1)$ is given by:

$$P(EDP > x \mid Sa(T_1) = sa,) = 1 - \Phi\left(\frac{\ln x - \hat{\mu}_{\ln EDP|Sa(T_1)=sa}}{\hat{\sigma}_{\ln EDP|Sa(T_1)=sa}}\right) \tag{3.3}$$

In Eqn. 3.3, $\hat{\mu}_{\ln EDP|Sa(T_1)=sa}$ and $\hat{\sigma}_{\ln EDP|Sa(T_1)=sa}$ are the sample mean and standard deviation for the *EDP*, respectively, and $\Phi(\cdot)$ is the standard normal cumulative distribution function. While maximum interstory drift has been found to be well represented by a lognormal distribution (Shome, 1999), a Kolmogorov-Smirnov test was developed to validate the use of this distribution for the case of the normalized plastic hysteretic energy, E_N, defined as:

$$E_N = \frac{E_H}{C_y D_y W} \tag{3.4}$$

where E_H is the total plastic hysteretic energy dissipated by the structure during the ground motion; D_y and C_y, the global displacement and seismic coefficient at first yield, respectively, (see Table 1); and W, the total weight of the structure. The seismic coefficient is defined as the base shear of the structure at first yield normalized by W. Figure 2a shows the normalized plastic hysteretic energy demands corresponding to an incremental dynamic analysis of frame F10. The figure suggests that the E_N distribution can be represented in a reasonable manner through a lognormal density function. To further illustrate this, figure 2b shows the E_N distribution corresponding to $Sa(T_I)$=1200cm/s². The K-S test for this data set suggests that E_N is well represented by a lognormal probability density function. Similar results were obtained for the other frames in a wide range of values of $Sa(T_I)$.

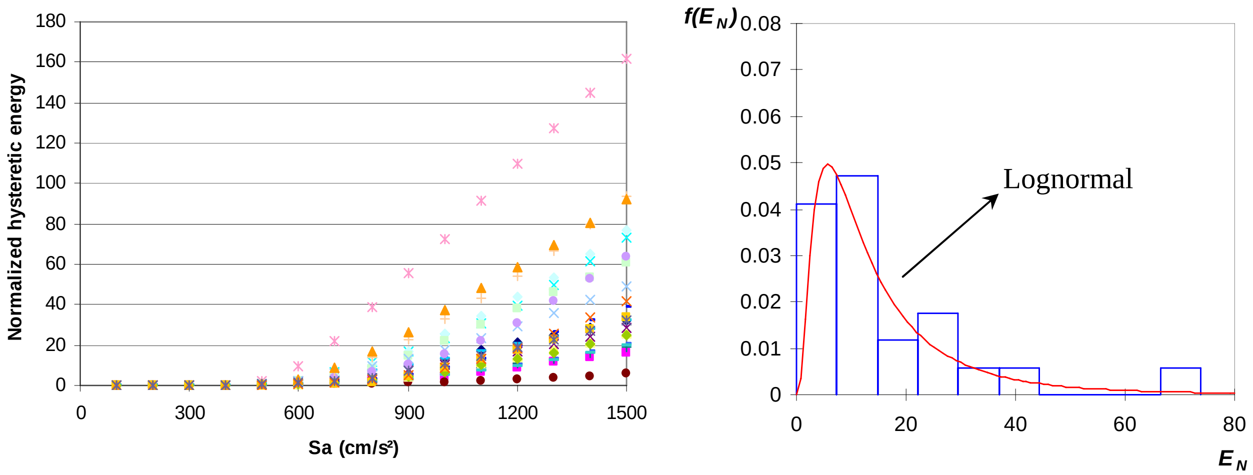

Figure 2 a) E_N demands from incremental dynamic analysis of frame F10; b) Probability density function for E_N , $Sa(T_I)$=1200 cm/s²

4. STRUCTURAL CAPACITY OF THE FRAMES

The structural reliability of the frames obtained in terms of maximum interstory drift and E_N can not be compared directly. To make this possible, the *EDPs* were normalized by their corresponding structural capacity. Particularly, the maximum interstory drift demand was normalized by its corresponding drift capacity, as indicated in Eqn 4.1, to establish a normalized damage measure in terms of this deformation parameter. While in the equation $I_{D\gamma}$ characterizes damage in terms of maximum interstory drift; γ_D and γ_C represent the demand and capacity of the structure in these terms. For the purposes of this study, a $\gamma_C = 0.03$ (recommended by the Mexican City building Code) was considered.

$$I_{D\gamma} = \frac{\gamma_D}{\gamma_C} \tag{4.1}$$

Similarly, a measure of damage in terms of normalized plastic hysteretic energy is formulated in Eqn. 4.2. While I_{DE_N} characterizes damage in terms of E_N; E_{ND} and E_{NC} represent the demand and capacity of the structure in these terms. E_{NC} can be estimated from Eqn. 4.3 (Bojórquez et al, 2008).

$$I_{DE_N} = \frac{E_{ND}}{E_{NC}} \tag{4.2}$$

$$E_{NC} = \frac{\sum_{i=1}^{NP} (2\, NC\, Z_f F_y \theta_{pa} F_{EHi})}{C_y D_y W} \tag{4.3}$$

In Eqn. 4.3, NP and NC are the number of stories and bays in the building, respectively; F_{EHi}, an energy participation factor; Z_f, the section modulus of the flanges of the elements; F_y, the yield stress; and finally, θ_{pa}, the cumulative plastic rotation capacity of the structural steel elements. This equation considers that the plastic energy is dissipated exclusively through the plastic behavior at both ends of the beams of the frames. Based on experimental test of steel elements subjected to cyclic loading, Akbas et al, (1997) found a wide range of values for θ_{pa}. From the point of view of Eqns. 4.1 and 4.2, failure corresponds to a value of one. A value of zero implies no damage has occurred in the structure.

5. COMPARISSON OF STRUCTURAL RELIABILITY

The ground motion seismic hazard curves corresponding to the SCT site in Mexico City (Alamilla, 2001) were used to evaluate the structural reliability of the frames in terms of the *EDPs* under consideration. A $\theta_{pa} = 0.10$ suggested by Akbas et al (1997) was adopted to evaluate the normalized plastic hysteretic energy capacity. Figure 3 shows the demand hazard curves for frame F8 obtained by formulating Eqn. 3.2 in terms of $I_{D\gamma}$ and I_{DE_N}. Values larger than one were plotted just for illustrative purposes (unity implies failure). Three zones can be appreciated in the figure. The first one corresponds to small values of I_D (see blue box), which would commonly be associated to the serviceability limit state. In this range of I_D, the mean annual rate of exceedance is larger for $I_{D\gamma}$ than for I_{DE_N}. This implies that for serviceability, the structural reliability is larger in terms of normalized plastic hysteretic energy, so that the design is controlled by maximum interstory drift. This is logical because the level of displacement control required by the serviceability conditions implies minimum or no plastic demands in the structural elements.

A second zone, corresponding to intermediate values of I_D (close to 0.5), can be noticed in figure 3 (orange box). In this zone, both demand hazard curves exhibit similar ordinates, implying that the design of frames for intermediate levels of damage is insensible to the measure of damage used to guarantee an adequate performance of the structure (maximum interstory drift or plastic hysteretic energy). Finally, a third zone (red box) can be appreciated for values of I_D close to one. Because this zone relates to structural failure, it is usually deemed as the most important. For design against failure, the process is controlled by cumulative plastic deformation demands, in this case represented by the normalized plastic hysteretic energy. This implies that cumulative plastic demands can have a very important role for seismic design, and that under some circumstances; an unsatisfactory design can be obtained if measures of the ground motion duration or of cumulative demands are not taken into account explicitly.

Although the importance of plastic cumulative demands for seismic design against failure has been explained with the help of figure 3, it is important to address the capacity of the structure in terms of E_N. Particularly, how does the value of θ_{pa} used in Eqn. 4.3 affect the reliability of the structure in terms of I_{DE_N}? And more specifically, if a larger value of θ_{pa} would have been used to establish the reliabilities in figure 3, would the conclusions change in terms of the importance of the plastic cumulative demands? Note that a value of 0.10 was considered before for θ_{pa}; and that if a larger value is adopted, the capacity of the structure in terms of cumulative plastic demands increases, in such manner that the reliability levels associated with the plastic energy demands can be similar or even larger than those associated with maximum interstory drift.

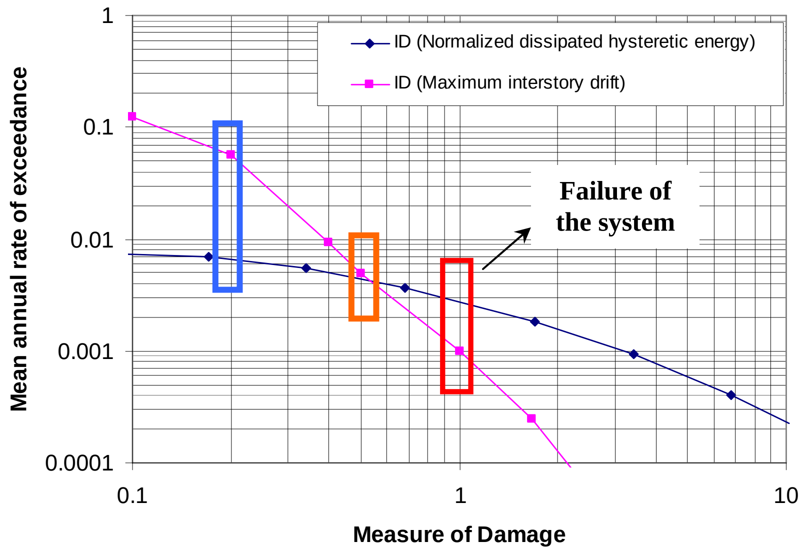

Figure 3 Hazard curves for frame F8 established in terms of $I_{D\gamma}$ and I_{DE_N}

Figure 4 compares fragility curves for frame F8 that are associated to failure in terms of maximum interstory drift and normalized plastic hysteretic energy ($\theta_{pa} = 0.34$). Both curves are practically the same, in such manner that the structural reliability of the frame is similar in terms of both *EDPs*. This implies that if the cumulative plastic rotation capacity of the steel beams is $\theta_{pa} = 0.34$, the maximum interstory drift is an adequate target parameter for seismic design against failure. Figure 5 shows experimental results of steel elements collected by Akbas et al (1997). The results were grouped to obtain a probability density function in terms of the cumulative plastic rotation capacity. A K-S test suggests that this function is well represented by a lognormal distribution, which is also included in the figure. Note that $\theta_{pa} = 0.34$ is larger than the median value obtained from the experimental data, and that only 29% of the specimens can reach a cumulative capacity that is equal or larger than 0.34. The above suggests that there is a need to control the plastic hysteretic energy demand or any other parameter related with cumulative demands when designing frame F8 against failure.

Table 2 shows the cumulative plastic rotation capacities required by the steel beams so that the structural reliability levels at failure of the different frames are similar in terms of both *EDPs* under consideration. In the case of frame F4, the results suggest that a design criterion based on maximum interstory drift limit is adequate. The θ_{pa} required for frame F6 is just a little larger than the median value observed in experimental tests, in such manner that a design based on maximum displacement demand could be unsatisfactory. For frames F8 and F10, the need to control cumulative plastic or normalized plastic hysteretic energy demands is clear.

Table 2 Cumulative plastic rotation required to achieve similar levels of reliability in terms of both *EDPs* under consideration

Frame	T_1 **(s)**	θ_{pa}
F4	0.90	0.21
F6	1.07	0.27
F8	1.20	0.34
F10	1.37	0.49

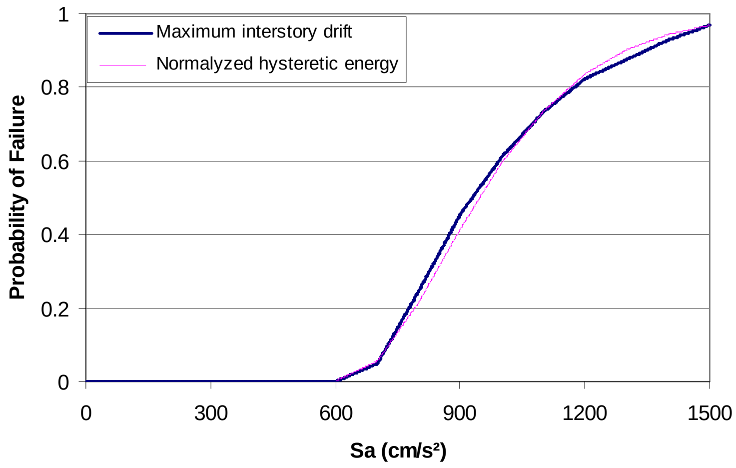

Figure 4 Fragility curves for frame F8 expressed in terms of maximum interstory drift and normalized plastic hysteretic energy, $\theta_{pa} = 0.34$

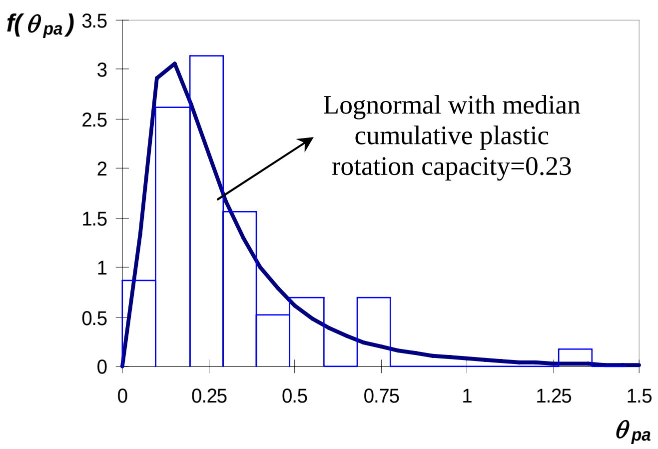

Figure 5 Probability density function for the cumulative plastic rotation capacity

6. CONCLUSIONS

The structural reliability of mid-rise steel buildings located on the Lake Zone of Mexico City was established in terms of two seismic demand parameters. One accounted for maximum deformation demands while the other did the same for cumulative plastic demands. For the serviceability limit state, seismic design is controlled by response parameters related to maximum deformation demands (e.g. maximum interstory drift). Although in the case of failure prevention, the structural reliability in terms of normalized plastic hysteretic energy is very sensitive to the cumulative plastic rotation capacity of the elements, it was observed that for frames designed according to the Mexico City Building Code and located in the Lake Zone of that city, the levels of reliability are smaller when cumulative damage is taken into account.

Under some circumstances, a seismic design based in the control of maximum interstory drift may exhibit actual structural reliability levels that are significantly smaller than those derived accounting for cumulative deformation demands. This is particularly true for structures with low cyclic capacity, and/or high cyclic capacity and having periods of vibration near to the dominant period of the soil. It is recommended to identify those cases in which a seismic design based on maximum demands are adequate, and those cases in which it is necessary to control response parameters that are clearly related to cumulative damage, such as the plastic hysteretic energy.

ACKNOWLEDGEMENTS

The study presented in this paper was developed within the activities of Rete dei Laboratori Universitari di Ingegneria Sismica – ReLUIS for the research program founded by the Dipartimento di Protezione Civile.

REFERENCES

Akbas, B. Shen, J. and Hao, H. (1997). Energy approach in performance-based design of steel moment resisting frames for basic safety objective. *The Structural Design of Tall Buildings* **10**, 193-217.

Alamilla, J. L. (2001). Criterios de diseño sísmico basados en confiabilidad para estructuras aporticadas. *Tesis presentada en la DEPFI para obtener el grado de Doctor en Ingeniería, UNAM*. (In Spanish)

Arroyo, D. and Ordaz, M. (2007). Hysteretic energy demands for SDOF systems subjected to narrow band earthquake ground motions. Applications to the lake bed zone of Mexico City", *Journal of Earthquake Engineering* **11**, 147-165.

Baker, J. W. and Cornell, C.A. (2005). A Vector-valued ground motion intensity measure consisting of spectral acceleration and epsilon", *Earthquake Engineering and Structural Dynamics* **34**, 1193-1217.

Baker, J. W. and Cornell, C.A. (2008). Vector-valued intensity measures for pulse-like near-fault ground motions. *Engineering Structures* **30:4**, 1048-1057.

Bojórquez, E. and Ruiz, S.E. (2004). Strength reduction factors for the valley of Mexico taking into account low cycle fatigue effects. *13º World Conference on Earthquake Engineering*, paper 516, Vancouver Canada.

Bojórquez, E., Ruiz S. E. and Terán-Gilmore A. (2008). Reliability-based evaluation of steel structures using energy concepts. *Engineering Structures* **30:6**, 1745-1759.

Bojórquez, E. Iervolino, I. and Manfredi, G. (2008). Evaluating a new proxy for spectral shape to be used as an intensity measure. 2008 Seismic Engineering International Conference Commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA'08).

Cornell, C.A. (1968). Engineering seismic risk analysis. *Bulletin of the Seismological Society of America*. **58:5**, 1583-1606.

Darwin, D. and Nmai, C.K. (1985). Energy dissipation in RC beams under cyclic loading. *Journal of Structural Engineering, ASCE*, **112:8**, 1829-1846.

Esteva, L. (1967). Criterios para la construcción de espectros para diseño por sismo. *Boletín del Instituto de Materiales y Modelos Estructurales*, **19**, Universidad Central de Venezuela. (In Spanish)

Rodríguez, M. and Ariztizabal, J. (1999), Evaluation of a seismic damage parameter. *Earthquake Engineering and Structural Dynamics* **28**, 463-477.

Shome, N. (1999). Probabilistic Seismic Demand Analysis of Nonlinear Structures. *Ph.D. Thesis, Stanford University*.

Terán-Gilmore, A. and Jirsa, J.O. (2007). Energy demands for seismic design against low-cycle fatigue. *Earthquake Engineering and Structural Dynamics* **36**, 383-404.

Terán-Gilmore, A. and Bahena-Arredondo, N. (2008). Cumulative ductility spectra for seismic design of ductile structures subjected to long duration motions: Concept and theoretical background. *Journal of Earthquake Engineering* **12**, 152-172.

Tothong, P. and Luco, N. (2007). Probabilistic seismic demand analysis using advanced ground motion intensity measures", *Earthquake Engineering and Structural Dynamics* **36**, 1837-1860.