Evaluation of Seismic Fragility of Steel Frames Using Vector-Valued IMs

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ABSTRACT:
The seismic fragility of steel frames subjected to narrow-band motions from the soft-soil of Mexico City is evaluated by means of different alternatives vector-valued ground motion intensity measures (IMs) comprised of two parameters. Because most of the seismic hazard maps around the world were developed for the spectral acceleration at first mode of vibration $Sa(T_1)$, all the vectors here considered are based in $Sa(T_1)$ as the first parameter. As the second parameter of the vector, the peak ground acceleration, peak ground velocity, duration, I_D, $R_{T1,T2}$ and N_p are considered. The maximum inter-story drift and a recently proposed energy-based damage index for steel frames were employed for structural performance evaluation. It is observed, as expected, that spectral-shape-based vector valued IMs are the best proxies for seismic fragility analysis reducing the uncertainties in the structural response.

Keywords: *seismic fragility analysis, vector-valued intensity measures, steel frames.*

1. INTRODUCTION

The earthquake ground motion potential is characterized by a parameter usually named the intensity measure (IM). One of the desirable features of an IM must be the ability to predict the response of a structure subjected to earthquakes (i.e., small variability of structural response given the IM). This ability is known as *efficiency*. Another desirable property is *sufficiency* which means that no other ground motion information is needed to characterize the structural response (Luco, 2002) and, in probabilistic structural assessment, allows decoupling the hazard and structural analysis. In the past, parameters as peak ground acceleration *PGA*, peak ground velocity *PGV*, Arias Intensity (Arias, 1970) and other were commonly used as IMs. More recently, the spectral acceleration at first mode of vibration of the structure, $Sa(T_1)$, has been thoroughly studied and became very popular, especially because it is the perfect predictor of the linear structural response of single degree of freedom system, and most of the worldwide seismic hazard maps quantify the seismic threat in terms of probability of exccedance of this parameter. Nevertheless, some limitations of $Sa(T_1)$ have been observed recently, and for this reason different researchers promote the use of vector-valued IMs (e.g., Bazzurro and Cornell, 2002; Baker and Cornell, 2005). Vector-valued IMs are based on the use of two or more parameters with the aim of predicting the response of a structure with more efficiency.

The aim of this paper is to evaluate the seismic fragility of steel framed structures subjected to narrow-band motions for the soft-soil of Mexico City, which is the largest city of Mexico and it is ranked as the eight richest cities in the world, by means of several vector-valued IMs. All the vector-valued IMs here considered are based on $Sa(T_1)$ as the first parameter. As the second parameter of the vector, *PGA*, *PGV*, ground motion duration t_D established according to Trifunac and Brady (1975) as the time interval delimited by the instants of time at which the 5% and 95% of the Arias Intensity occurs, the I_D factor proposed by Cosenza and Manfredi (1997), the $R_{T1,T2}$ (Cordova et al., 2001) and N_p (Bojórquez and Iervolino, 2009) were used. Note that the vector-valued IMs were selected to account for maximum and cumulative damage potential. Moreover, the use of $Sa(T_1)$ as the first parameter is because studies have found the sufficiency of this IM with respect to magnitude and distance (Shome, 1999; Iervolino and Cornell, 2005). Finally, the seismic fragility is evaluated for two different performance parameters: a) the maximum inter-story drift which is the most common parameter in the seismic design codes to control the structural behavior, and b) an energy-based damage index for steel frames recently proposed (Bojórquez et al., 2009a, 2010).

2. METHODOLOGY

2.1 Vector-valued ground motion IMs selected

The seismic fragility is assessed through six different vector-valued ground motion IMs. Table 2.1 summarizes the principal characteristics considered in each IM (e.g. peak response, duration and spectral shape). The first two IMs are $<Sa(T_1), PGA>$ and $<Sa(T_1), PGV>$ which are representative of peak responses. The second two IMs are $<Sa(T_1), t_D>$ and $<Sa(T_1), I_D>$ to represent the cumulative damage potential and the influence of ground motion duration, where the I_D factor is defined as:

$$I_D = \frac{\int_0^{t_F} a(t)^2\,dt}{PGA \cdot PGV} \tag{2.1}$$

In equation 2.1 $a(t)$ is the acceleration time-history and t_F is the total duration of the ground motion.

The last two IMs considered are $<Sa(T_1), R_{T1,T2}>$ and $<Sa(T_1), N_p>$. These are representative of the spectral shape, which has been recently proposed as the main ground motion feature expressing the earthquake structural potential. While $R_{T1,T2}$ is the ratio between the spectral acceleration at period T_2 divided by spectral acceleration at period T_1; N_p is defined in Eqn 2.2. The information given by this equation is that if we have one or n records with a mean N_p value close to one, we can expect that the average spectrum to be about flat in the range of periods between T_1 and T_N. For a value of N_p lower than one it is expected an average spectrum with negative slope beyond T_1. In the case of N_p values larger than one, the spectra tend to increase beyond T_1. Finally, the normalization between $Sa(T_1)$ let N_p be independent of the scaling level of the records based on $Sa(T_1)$, but most importantly it helps to improve the knowledge of the path of the spectrum from period T_1 until T_N, which is related with the nonlinear structural response. In this study, a value of T_2 equal to twice the first mode period was chosen, because Cordova et al. (2001) and Baker (2005) identify it as adequate, and Bojórquez et al. (2008b) confirm this for nonlinear SDOF systems and considering different performance parameters. Finally, Bojórquez and Iervolino observed that the value of T_N around 2 or 2.5 times T_1 seems adequate.

$$N_p = \frac{Sa_{avg}(T_1 \ldots T_N)}{Sa(T_1)} \tag{2.2}$$

Table 2.1. Summary of the vector-valued IMs considered.

Intensity Measure	Peak response	Duration	Spectral shape
$<Sa(T_1), PGA>$	*		
$<Sa(T_1), PGV>$	*		
$<Sa(T_1), t_D>$		*	
$<Sa(T_1), I_D>$	*	*	
$<Sa(T_1), R_{T1,T2}>$	*		*
$<Sa(T_1), N_p>$	*		*

2.2 Performance parameters

The seismic fragility of steel frames is obtained in terms of maximum inter-story drift and an energy based damage index. However, with the aim to compare both parameters, it is necessary to normalize the maximum inter-story drift by its corresponding drift capacity to establish a normalized damage measure in terms of this deformation parameter, this is illustrated in the following equation:

$$D_{\gamma} = \frac{\gamma_D}{\gamma_u} \tag{2.3}$$

where D_{γ} characterizes damage in terms of maximum inter-story drift; and γ_D and γ_u represent the demand and capacity of the structure, respectively. For the purposes of this study, γ_u equal 0.05 was considered for the steel frames analyzed (Bojórquez et al., 2009b).

Similarly, a measure of damage in terms of normalized plastic hysteretic energy can be formulated as (Bojórquez et al., 2009a, 2010):

$$D_{EN} = \frac{E_{ND}}{E_{NC}} \tag{2.4}$$

where D_{EN} characterizes damage in terms of normalized plastic hysteretic energy; and E_{ND} and E_{NC} represent the demand and capacity of the structure in these terms, respectively. Within this context, E_{ND} for a particular frame is estimated as the sum of the plastic hysteretic energy dissipated by all its structural members. E_{NC} can be estimated as (Akbas et al., 1997; Bojórquez et al., 2008a):

$$E_{NC} = \frac{\sum_{i=1}^{NS} (2\, NB\, Z_f F_y \theta_{pa} F_{EHi})}{C_y D_y W} \tag{2.5}$$

where NS and NB are the number of stories and bays in the building, respectively; F_{EHi}, an energy participation factor that accounts for the different contribution of each story to the energy dissipation capacity of a frame; 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 plastic behavior at both ends of the beams of the frames. A $\theta_{pa} = 0.23$ was used to characterize the normalized plastic hysteretic energy capacity at the ends of the beams of the frames (Bojórquez et al., 2009b).

From a physical point of view, Eqn. 2.4 represents a balance between the structural capacity and demand in terms of energy. In this sense, this formulation follows the direction initially established by Housner in 1956 for an energy-based design. Note that in both damage measures, a value of 1 implies the structural failure.

2.3 Structural steel frame models and seismic records

Two regular steel frames designed according to the Mexico City Building Code were subjected to 23 soft-soil long duration ground motions recorded in the Lake Zone of Mexico City which have a dominant period (T_s) of two seconds. The frames, which were assumed to be used for office occupancy, have three bays of 8m and a number of levels of eight and ten with a story height of 3.5m. The frames were designed for ductile detailing. A36 steel and W sections were used for the beams and columns of the frames. An elastic-plastic model with 3% strain-hardening was considered to model the cyclic behaviour of the steel members (Bojórquez and Rivera, 2008). The critical damping ratio was assumed equal to 3%. Relevant characteristics for each frame, such as the fundamental period of vibration (T_1), and the seismic coefficient and displacement at yielding (C_y and D_y) are shown in Table 2.2 (the latter two values were established from pushover analyses). The earthquake motions were recorded during seismic events with magnitudes of seven or larger, and having epicenters located at distances of 300 km or more from Mexico City. Some important characteristics of the records are summarized in Table 2.3.

Table 2.2. Relevant characteristics of the steel frames

Frame	Number of Stories	T_1 (s)	C_y	D_y(m)
F8	8	1.20	0.38	0.192
F10	10	1.37	0.36	0.226

Table 2.3. Seismic records

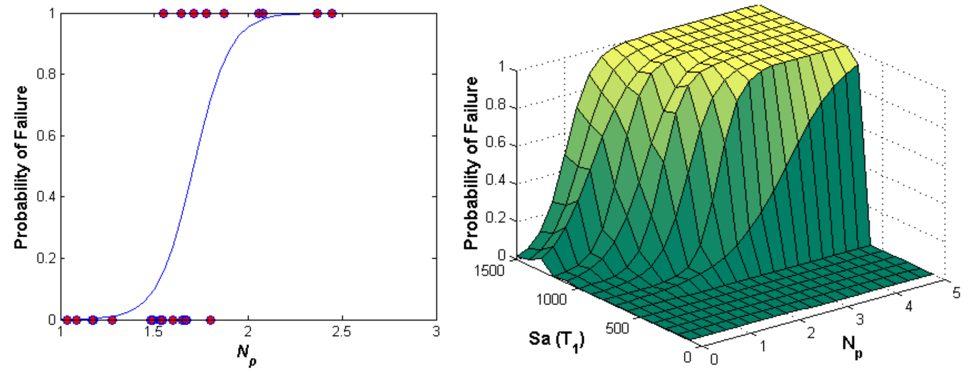
Record	Date	Magnitude	Station	*PGA* (cm/s²)	*PGV* (cm/s)
1	25/04/1989	6.9	Alameda	45.0	15.6
2	25/04/1989	6.9	Garibaldi	68.0	21.5
3	25/04/1989	6.9	SCT	44.9	12.8
4	25/04/1989	6.9	Sector Popular	45.1	15.3
5	25/04/1989	6.9	Tlatelolco TL08	52.9	17.3
6	25/04/1989	6.9	Tlatelolco TL55	49.5	17.3
7	14/09/1995	7.3	Alameda	39.3	12.2
8	14/09/1995	7.3	Garibaldi	39.1	10.6
9	14/09/1995	7.3	Liconsa	30.1	9.62
10	14/09/1995	7.3	Plutarco Elías Calles	33.5	9.37
11	14/09/1995	7.3	Sector Popular	34.3	12.5
12	14/09/1995	7.3	Tlatelolco TL08	27.5	7.8
13	09/10/1995	7.5	Cibeles	14.4	4.6
14	09/10/1995	7.5	Córdoba	24.9	8.6
15	09/10/1995	7.5	Liverpool	17.6	6.3
16	09/10/1995	7.5	Plutarco Elías Calles	19.2	7.9
17	11/01/1997	6.9	CU Juárez	16.2	5.9
18	11/01/1997	6.9	Centro urbano Presidente Juárez	16.3	5.5
19	11/01/1997	6.9	García Campillo	18.7	6.9
20	11/01/1997	6.9	Plutarco Elías Calles	22.2	8.6
21	11/01/1997	6.9	Est. # 10 Roma A	21.0	7.76
22	11/01/1997	6.9	Est. # 11 Roma B	20.4	7.1
23	11/01/1997	6.9	Tlatelolco TL55	13.4	6.5

2.4 Evaluation of seismic fragility using vector-valued IMs

The advantages of the evaluation of seismic fragility via vector-valued ground motion IMs, instead of scalar IMs, should be related to a more accurate estimation of the probability of failure. Herein, seismic fragility assessment for the vector case is developed by nonlinear incremental dynamic analysis of the frames subjected to the records by using the first parameter of the vector, in this case $Sa(T_1)$, and then using logistic regression (Baker, 2005) to fit failure (F) and non failure cases for the second parameter. It is important to note that the records are scaled at a fixed value of $Sa(T_1)=x$ and for this specific value, the logistic regression is applied for the second parameter. The probability of failure P_F using logistic regression is obtained as follows:

$$P_F(F \mid S_a(T_1) = x_1, I_{M2} = x_2) = \frac{1}{1 + e^{(-\beta_1 - \beta_2 x_2)}} \tag{2.6}$$

where I_{M2} is the second parameter of the vector, β_1 and β_2 are coefficients obtained from regression analysis of the results for the records scaled at a fixed $Sa(T_1)=x_1$, which means that the coefficients change for different scaling levels. Fig. 2.1a illustrates an example of logistic regression for a fixed spectral acceleration, and for the vector $<Sa(T_1), N_p>$, and Fig. 2.1b presents an example of the probability of failure as a function of $Sa(T_1)$ and N_p. Figure 2.1b shows good correlation between N_p and the probability of failure, and the logistic relation illustrates the accuracy in the prediction of probability of failure characterized by the increasing in the probability of failure as N_p increase. In particular, there are only few values of N_p for which the probability of failure is zero and one.

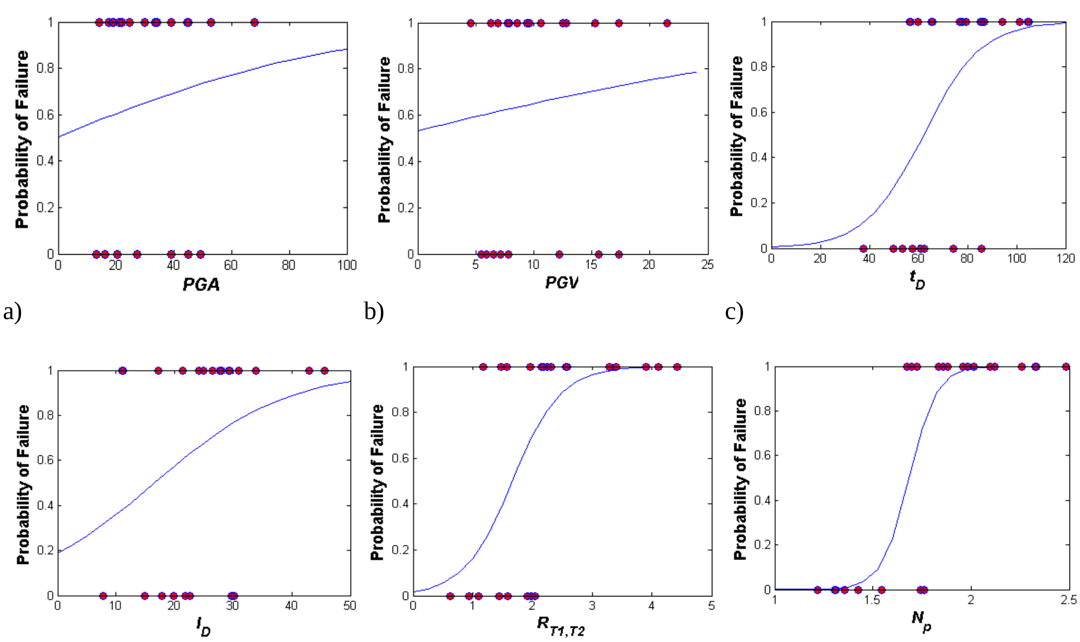


a) b)

Figure 2.1. a) Logistic regression for a fixed spectral acceleration and the vector <$Sa(T_1)$, N_p>; b) probability of failure versus, $Sa(T_1)$ and N_p

3. RESULTS AND DISCUSSION

The results of the analyses for the two frames considered, the different ground motion IMs, and performance parameters are presented in this section. First, the results of logistic regression for frames F8 and F10 and all the IMs considered are illustrated in Fig. 3.1 and 3.2 for D_γ and for a fixed spectral acceleration $Sa(T_1)$=1500cm/s², but similar results, not included here, are valid for other scaling levels. The results show the relation between all the IMs considered and the probability of failure. For frame F8, *PGA* and *PGV* seems less explicative with respect to failure prediction given spectral acceleration as expected, while parameters as t_D, I_D and $R_{T1,T2}$ are better related; however, N_p results very effectiveness and more appropriate with respect to the IMs here considered to predict the probability of failure with lower uncertainty (i.e., less flat regression curves). Note that in Fig. 3.1e and 3.2e only one or two values of N_p have both possibilities of probability of failure. This is not valid for other parameters, in particular, Fig. 3 illustrate that the probability of failure is zero or one for similar values of *PGA*, *PGV*, t_D, I_D or $R_{T1,T2}$. Finally, Fig. 3.3 suggests that N_p also can be used to predict the probability of failure for frame F8 in terms of D_{EN}, a similar conclusion is valid for frame F10 which is not included herein.



a) b) c)

d) e) f)

Figure 3.1. Comparing probability of failure in terms of D_γ using logistic regression for frame F8 at a fixed $Sa(T_1)$=1500cm/s² versus a) *PGA*; b) *PGV*; c) t_D; d) I_D; e)$R_{T1,T2}$ and f) N_p

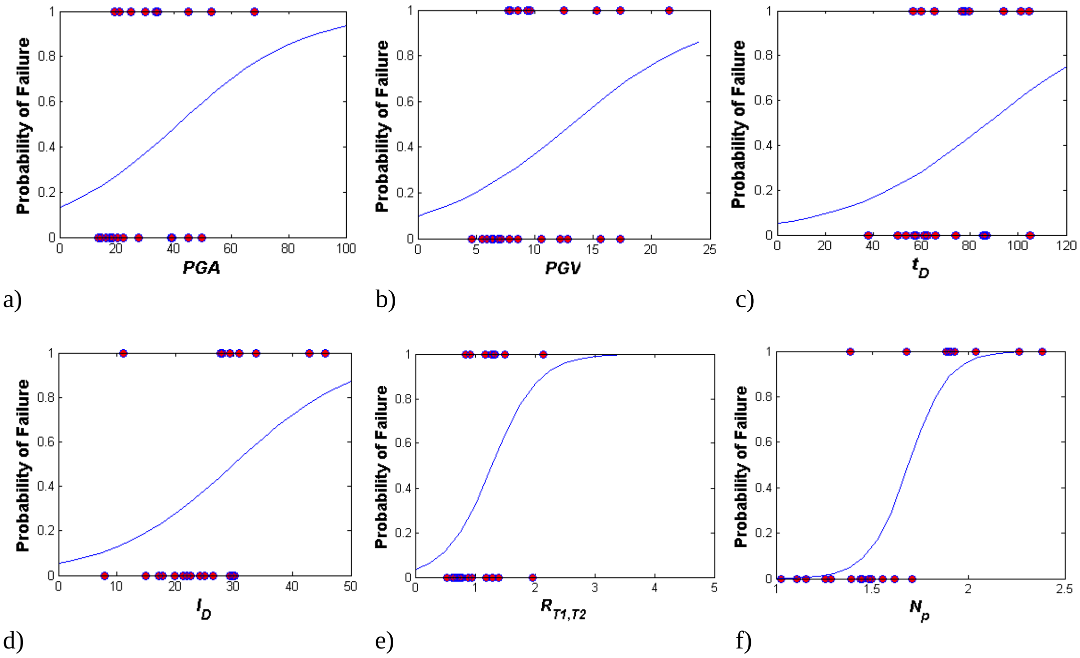


Figure 3.2. Comparing probability of failure in terms of D_γ using logistic regression for frame F10 at a fixed $Sa(T_1)$=1500cm/s² versus a) PGA; b) PGV; c) t_D; d) I_D; e)$R_{T1,T2}$ and f) N_p

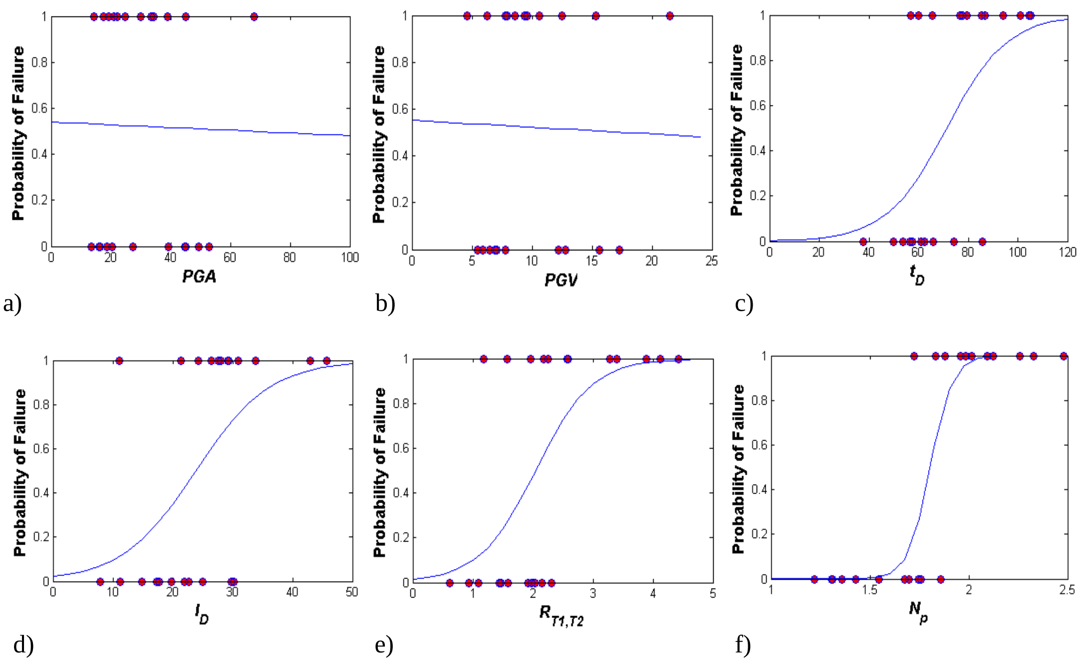
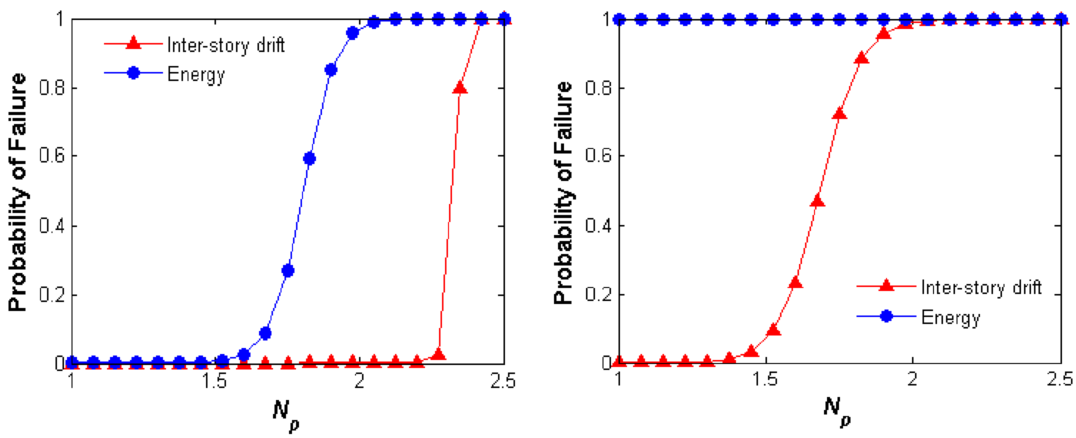


Figure 3.3. Comparing probability of failure in terms of D_{EN} using logistic regression for frame F8 at a fixed $Sa(T_1)$=900cm/s² versus a) PGA; b) PGV; c) t_D; d) I_D; e) $R_{T1,T2}$ and f) N_p

3.1 Comparing probability of failure for D_γ versus D_{EN}.

The probability of failure in terms of both performance parameters considered is analyzed in this section. For the sake of brevity, only the comparison for the vector $<Sa(T_1), N_p>$ is illustrated. This vector was chosen because it was the best ground motion intensity measure to predict the probability of failure in term of maximum inter-story drift and energy demands for the cases here studied. Fig. 3.4

compares for frame F8 and two scaling levels the probability of failure for drifts and hysteretic energy. Larger probability of failure in terms of the energy based-damage index in the complete range of N_p values are observed, which suggests that if only maximum inter-story drift is considered as the main performance parameter to achieve adequate seismic design of buildings, the cumulative demands through plastic deformation could be underestimated for the case of steel structures located in the soft-soil of Mexico City, in such way that it is important to incorporate in future seismic design codes information to characterize the effects of cumulative demands, otherwise, the seismic designs could be unsatisfactory.



a) b)

Figure 3.4. Probability of failure for D_γ versus D_{EN} for frame F8 at a fixed a) $Sa(T_1)$=900cm/s²; b) $Sa(T_1)$=1500cm/s²

4. CONCLUSIONS

The evaluation of probability of failure of steel frames subjected to narrow-band motions from Mexico City was developed by means of different vector-valued ground motion intensity measures. The results suggest that $<Sa(T_1), N_p>$ is the best proxy to predict the structural failure in terms of maximum inter-story drift, and normalized hysteretic energy among the IMs compared. In fact, $<Sa(T_1), N_p>$ could be a good candidate for the next generation of ground motion intensity measures. Finally, the probability of failure for D_γ versus D_{EN} was compared for similar scaling levels in terms of the vector $<Sa(T_1), N_p>$. It is observed larger probability of failure of the structural frames with the use of the energy-based damage index. This is valid for different scaling levels, which may imply the importance to consider some parameter to characterize the ground motion duration effect or cumulative demand influence in seismic fragility assessment.

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