A Spectral Shape-based Scalar Ground Motion Intensity Measure for Maximum and Cumulative Structural Demands

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ABSTRACT:
In this paper, a new spectral-shape-based scalar ground motion intensity measure (IM) to estimate maximum and cumulative demands is discussed. The parameter is based on the pseudo-acceleration response spectrum. In the paper it is shown that it is *efficient* with respect to the estimation of structural response. Moreover, it is better correlated with maximum and cumulative structural response of single degree of freedom (SDOF), and multi degree of freedom (MDOF) structures subjected to seismic records with different characteristics (ordinary, near-fault pulse-like and long-duration narrow-band), if compared with the most popular IMs. Finally, it is demonstrated that probabilistic seismic hazard analysis for the parameter proposed can be performed with already available tools.

Keywords: *spectral shape, scalar intensity measures, vector-valued intensity measures, cumulative demand.*

1. INTRODUCTION

Several studies suggest that spectral shape is the main ground motion feature expressing the earthquake structural potential. In particular, ground motion intensity measures which represent the spectral shape in a range of periods are considered the best proxies in structural seismic demand analysis, at least if the displacement response is of interest. For this reason, recently, scalar and vector-valued intensity measures, based on spectral shape, have been proposed (Bazzurro and Cornell 2002; Baker and Cornell 2005; Baker and Cornell 2008; Bojórquez and Iervolino, 2009). A desirable property of an IM is *sufficiency* (i.e., the structural response does not depend on other ground motion features given the IM) which is important in probabilistic structural assessment to decouple the hazard and structural analysis (Bazzurro, 1998; Shome, 1999; Luco, 2002; Iervolino and Cornell, 2005). Another feature, which a good IM has to show, is the ability to predict the structural response, which is known as *efficiency* (i.e., the structural response is well correlated with the IM); e.g., Bazzurro, 1998; Shome, 1999; Luco, 2002. Vector-valued IMs are used to achieve sufficiency and efficiency at the same time and this is done including in the vector two or more parameters representative of ground motion. Nevertheless, scalar IMs are simpler to use and manage. The aim of this study is to investigate a spectral-shape-based scalar ground motion intensity measure based on the pseudo-acceleration which has comparable capabilities of vector-valued IMs. It will be shown that the IM here discussed is efficient to predict the maximum and cumulative structural response of nonlinear SDOF, and steel frame structures subjected to ordinary, near-fault pulse-like and long-duration narrow-band seismic records.

2. *Np* A PARAMETER TO CHARACTERIZE THE SPECTRAL SHAPE

It has been discussed that IMs often try to capture the structural response via the spectral shape with different degrees of success. For example, $Sa(T_1)$, is the perfect predictor for the response of elastic SDOF systems, and a good predictor for elastic multi-degree of freedom (MDOF) systems dominated by the first mode of vibration, associated to the T_1 period, and some studies have found the sufficiency of $Sa(T_1)$ with respect to magnitude and distance (Shome, 1999; Iervolino and Cornell, 2005). Nevertheless, $Sa(T_1)$ does not provide information about the spectral shape in other regions of the spectrum, which may be important for the nonlinear behavior (beyond T_1) or for structures dominated by higher modes (before T_1). In the case of structures with nonlinear behavior, the structural vibration period is elongated, as sketched in Fig. 2.1. As a consequence, the structure may be sensitive to different spectral values associated to the whole range, from the fundamental period until the maximum elongated period of the structure T_N. For this reason, it is important that the intensity measures provide information about the spectral shape in a whole period interval.

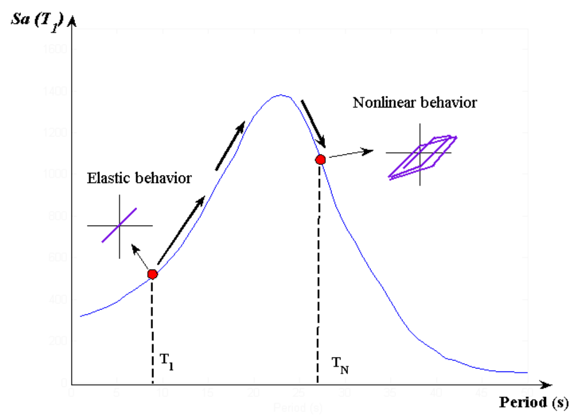


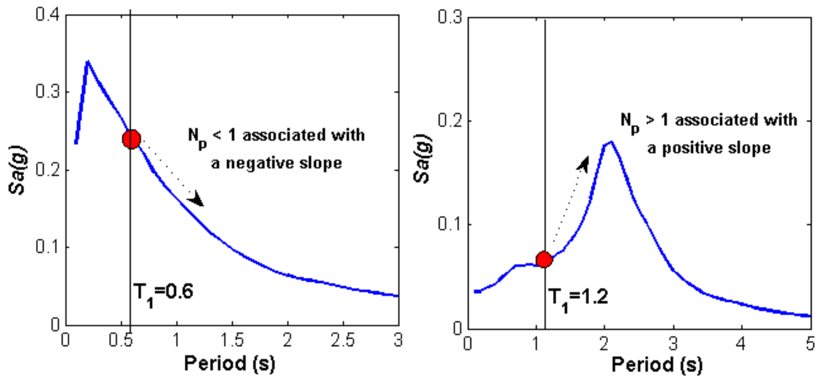
Figure 2.1. Elongated period T_N due to nonlinear behavior.

Although parameters as $Sa_{avg}(T_1...T_N)$ or the area under the spectrum, account for the spectral shape, a specific value of $Sa_{avg}(T_1...T_N)$, or the area under the spectrum, may be associated to different patterns of the spectrum between T_1 and T_N, that is, with different spectral shapes. A useful, yet simple, improvement may be use the geometrical mean $Sa_{avg}(T_1...T_N)$ but normalizing it by $Sa(T_1)$. To this aim a new parameter named N_p (Eqn. 2.1) may be introduced (Bojórquez and Iervolino, 2009).

$$N_p = \frac{Sa_{avg}(T_1...T_N)}{Sa(T_1)} \tag{2.1}$$

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 N_p lower than one it is expected an average spectrum with negative slope. As an example, the mean value of N_p for a group of 191 ordinary records in the period $T_1 = 0.6$s is 0.39 and the period $T_N=2T_1$, this value will be discussed below. In Fig. 2.2a, the average spectrum of this set is illustrated. In the case of N_p values larger than one, the spectra tend to increase beyond T_1. As it can be appreciated for a set of 31 narrow-band records, where the mean value of N_p=1.9 for T_1=1.2s and T_N=2T_1, the average spectrum shows an increasing accelerations zone (see Fig. 2.2b). 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 about the path of the spectrum from period T_1 until T_N, which is related with the nonlinear structural response.

It is important to note that N_p can be generalized for spectral shapes based on other parameters, for example: elastic displacement, inelastic displacement, velocity, and others. Herein, however, only N_p computed on acceleration spectra is analyzed because: a) several studies support the use of the elastic pseudo-acceleration spectral shape for the estimation of structural response based on maximum displacement (Bazzurro and Cornell 2002; Baker and Cornell 2005; Baker and Cornell 2008; Bojórquez and Iervolino, 2009); b) it is simple to compute, which is important for practical application purposes; and c) probabilistic seismic hazard analysis can be performed with tools currently available. An important issue that must be addressed is the definition of T_N. In this study for all the cases analyzed a T_N equals to $2T_1$ was used; see Bojórquez and Iervolino (2009).



a) b)

Figure 2.2. Average elastic response spectra for a set of: a) ordinary records with N_p=0.39, b) narrow-band records with N_p=1.9.

3. SCALAR GROUND MOTION IMs BASED ON N_p

Bojórquez and Iervolino (2009) introduced a vector-valued IM based on $Sa(T_1)$ and N_p $<Sa,N_p>$, which results more efficient than the most commonly used IMs. However, the use of the vector-valued intensity measure, with the aim of developing probabilistic seismic demand analysis of a structure, requires the evaluation of conditional distributions of N_p given $Sa(T_1)$. On the other hand, in the case of scalar IMs, the probabilistic seismic demand analysis can be carried out more easily. Moreover, the relationship with the structural response is clearer when using scalar IMs. Therefore, herein a scalar ground motion IM based on $Sa(T_1)$ and N_p, with similar characteristics to the IM proposed by Cordova et al. (2001), is considered:

$$I_{Np} = Sa(T_1)\, N_p^{\ \alpha} \tag{3.1}$$

In Eqn. 3.1, I_{Np} is the scalar ground motion intensity measure proposed, and α value has to be calibrated depending on the structure and the earthquake demand parameter considered. From Eqn. 3.1, it is possible to note that: a) the spectral acceleration at first mode of vibration is a special case of I_{Np}, and this occurs when α is equals with zero; b) $Sa_{avg}(T_1 \ldots T_N)$ also corresponds to the particular case when $\alpha = 1$; and c) the IM proposed by Cordova et al. (2001) also correspond to a particular case when only two point of the spectrum are considered T_1 and T_N, which implies that the geometrical mean is evaluated as $Sa_{avg}(T_1,T_N)$. Analyses below suggest that the optimal values of α are in a range from zero to one, which means giving different weights to the spectral ordinates beyond the first mode compared with the spectral acceleration at T_1.

4. SEISMIC RECORDS, STRUCTURAL MODELS AND EARTHQUAKE DEMAND PARAMETERS

To explore the intensity measure introduced, three sets of 31 seismic records were considered to represent ordinary, near-fault pulse-like and long-duration narrow-band motions. The set of ordinary records is a subset selected randomly of the set collected by Tothong (2007). The ground motion records were originally obtained from the NGA database. The closest distance to fault rupture is between 15km and 95km, and the moment magnitude ranges from 5.65 to 7.90.

The pulse-like records consist of a set of 31 motions with a pulse period close to 1s (mean $T_p \approx 1.0$s). To warrant that they correspond to near-source pulse-like records, they were rotated in the fault-normal direction. The earthquake magnitude range is from 5.6 to 7.6, and the closest distance to fault rupture less than 22km. The pulse-like set is a subset of that in Iervolino and Cornell (2008).

Long-duration narrow-band motions have the special characteristic of affecting specific structures considerably in a short range of periods (especially those with softening effect or vibration periods near to the soil period). In fact, these records demand large energy dissipation capacity in structures if compared to broad-band motions (Terán and Jirsa, 2007). A special case where long-duration narrow-band motions are observed corresponds to Mexico City. In this study, a set of 31 ground motion records obtained from the soft-soil of the Valley of Mexico was used. The magnitude range is 6.9 to 8.1. The records, previously used in Bojórquez et al. (2008), correspond to the far field and they were selected for a soil period close to 2 seconds, corresponding to most of the damages in buildings caused by the 1985 Mexican Earthquake. The average duration of the records is equal to 73.1s obtained according to Trifunac and Brady (1975).

Regarding structural models, nonlinear SDOFs, and a steel moment resisting frame are analyzed. The SDOFs have vibration periods T=0.5s and T=1s, bilinear hysteretic behavior, 5% of post yielding stiffness and 5% damping ratio. The steel frame designed with the requirements of the Mexican Building Code was used in a previous study Bojórquez et al. (2007). The first mode vibration period is 1.20, and it has 3 bays and 8 stories. The height of all the stories is 3.5m and the width of the bays is 8m. A bilinear hysteretic behavior with 3% of post yielding stiffness and 3% of critical damping is used. Table 1 summarizes the principal properties of each model.

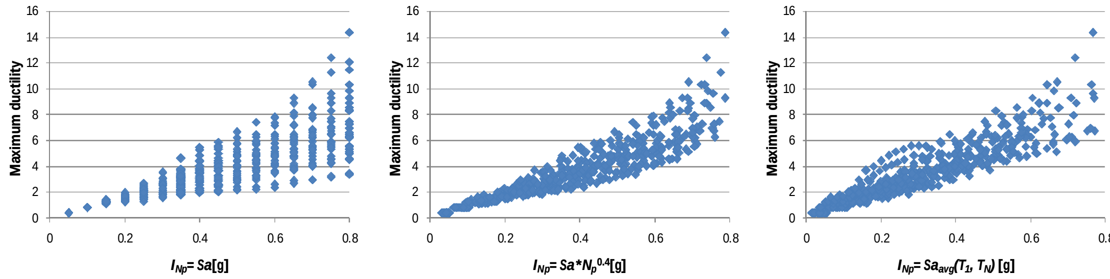
Finally, the earthquake demand parameters EDPs considered are the ductility displacement demand μ_D (ratio of maximum displacement D_m divided by the yielding displacement D_y), the maximum interstory drift ratio and the dissipated hysteretic energy (E_N) normalized by displacement and yielding strength (F_y). For MDOF systems, F_y (yielding base shear) and D_y were obtained from a push-over analysis, and the dissipated hysteretic energy corresponds to the total plastic energy dissipated by the structure (the plastic energy dissipated by all the elements). The ductility was evaluated as the ratio between the maximum displacement at the roof and the yielding displacement.

Table 1. Summary of the structural models.

Structural Model	Period T_1 [s]	Hysteretic Behavior Model	Damping [% of critical]	Subjected to ground motions
SDOF	0.5	Bilinear	5%	Pulse-like
SDOF	1	Bilinear	5%	Ordinary
8 stories steel frame	1.20	Bilinear	3%	Long-duration

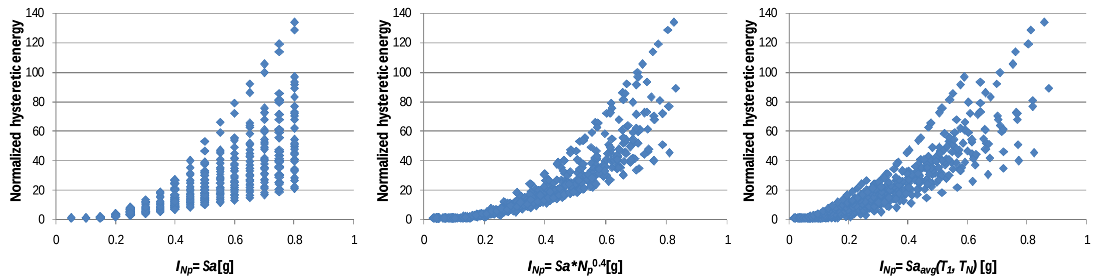
5. RESULTS

With the aim to show the effectiveness of the spectral-shape-based scalar ground motion intensity measure proposed, the SDOF system with T=1s was subjected to the set of ordinary records. I_{Np} is evaluated for different values of α. As it has been discussed in section 3, α=0 represents the case of $Sa(T_1)$ and α=1 corresponds to $Sa_{avg}(T_1...T_N)$. First, nonlinear incremental dynamic analyses were developed by using $Sa(T_1)$ and then, the results are compared with the associated values of I_{Np} for different α. Fig. 5.1 compares for different values of α, I_{Np} versus maximum ductility. The analyses results suggest that the optimal values of α are smaller than one and larger than zero. In general, I_{Np} appears more efficient than $Sa(T_1)$ and, at least, of comparable efficiency with respect to $Sa_{avg}(T_1...T_N)$. Similar results are observed for this structure compared with the normalized hysteretic energy, but in this case I_{Np} also results more efficient than $Sa_{avg}(T_1...T_N)$ (see Fig. 5.2).



a) b) c)

Figure 5.1. Relationships between I_{Np} and maximum ductility in the SDOF with T=1s subjected to ordinary records for: a) α=0, b) α=0.4, c) α=1

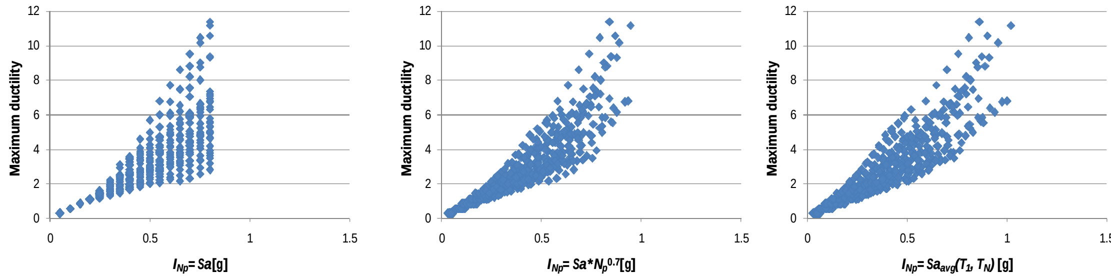


a) b) c)

Figure 5.2. Relationships between I_{Np} and normalized hysteretic energy in the SDOF with T=1s subjected to ordinary records for: a) α=0, b) α=0.4, c) α=1

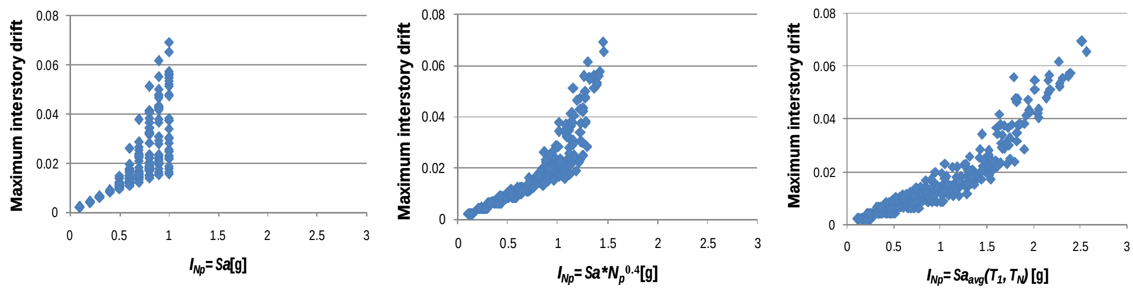
It was observed that near-fault pulse-like records may be threatening for structures where the relationship between the fundamental period and that of the pulse of the record is about one half (T_1/T_p = 0.5) (Alavi and Krawinkler, 2006; Chiocarelli and Iervolino, 2010). Then, it is expected that a SDOF system with T = 0.5s subjected to the set of 31 near-fault motions has unusual inelastic demand. Fig. 5.3 illustrates the relationship between maximum ductility and I_{Np} for different values of α and for the SDOF with T=0.5s excited by the set of pulse-like records. Generally, a good relationship between I_{Np} and the structural response is observed. As in the case of ordinary records, α play an important role in the efficiency. Similar results are found for the steel frame subjected to the set of long-duration narrow-band motions, the results to which are illustrated in Fig. 5.4. I_{Np} achieves the best accuracy and efficiency for intermediate values larger than zero and smaller than one. This conclusion holds in the case of normalized hysteretic energy demands as it is shown in Fig. 5.5 for the steel frame under the long-duration narrow-band motions.

In general, I_{Np} seems to improve the efficiency in terms of maximum and cumulative structural demands if compared with the most common IMs (spectral acceleration at first mode of vibration) and it is comparable to $Sa_{avg}(T_1 ... T_N)$. I_{Np} has the advantage that it is possible to optimize its capabilities via α and T_N. Regarding this latter parameter see Bojórquez and Iervolino (2009).



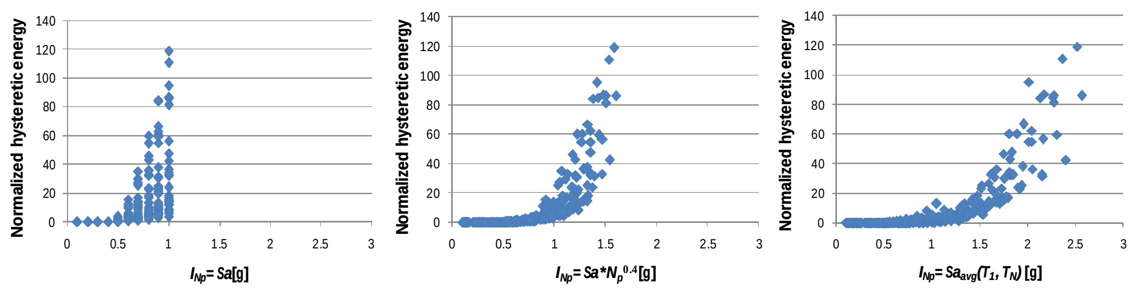
a) b) c)

Figure 5.1. Relationships between I_{Np} and maximum ductility in the SDOF with T=0.5s subjected to near-fault pulse-like records for: a) α=0, b) α=0.7, c) α=1



b) b) c)

Figure 5.4. Relationships between I_{Np} and maximum interstory drift in the steel frame subjected to narrow-band records for: a) α=0, b) α=0.4, c) α=1



a) b) c)

Figure 5.5. Relation between I_{Np} and normalized hysteretic energy in the steel frame subjected to narrow-band records for: a) α=0, b) α=0.4, c) α=1

6. PROBABILISTIC SEISMIC HAZARD ANALYSIS FOR I_{Np}

Because the actual possibility of computing hazard analysis is crucial for the applicability of any proposed IM in this section it is shown how it can be performed for I_{Np} with ordinary tools currently available for other ground motion intensity measures. In fact, substituting Eqn. 2.1 in Eqn. 3.1 and applying the natural logarithm, it results as:

$$\ln\left(I_{Np}\right) = \ln\left[Sa(T_1)\right] + \alpha \ln\left[\frac{Sa_{avg}(T_1 \ldots T_N)}{Sa(T_1)}\right] \tag{6.1}$$

Since $Sa_{avg}(T_1 ... T_N)$ is obtained by Equation 6.2, the mean and variance of $\ln(I_{Np})$ given in Equation 6.1 can be expressed as Equation 6.3 and Equation 6.4.

$$Sa_{avg}(T_1 ... T_N) = \sqrt[N]{\prod_{i=1}^{N} Sa(T_i)} \tag{6.2}$$

$$\mathrm{E}\left[\ln\left(\mathrm{I_{Np}}\right)\right] = (1-\alpha)\,\mathrm{E}\left\{\ln\left[\mathrm{Sa}(\mathrm{T_1})\right]\right\} + \frac{\alpha}{\mathrm{N}}\sum_{\mathrm{i}=1}^{\mathrm{N}} \mathrm{E}\left\{\ln\left[\mathrm{Sa}(\mathrm{T_i})\right]\right\} \tag{6.3}$$

$$\begin{aligned} Var\left[\ln\left(I_{Np}\right)\right] &= \alpha^2 Var\left\{\ln\left[Sa_{avg}(T_1 ... T_N)\right]\right\} + (1-\alpha)^2 Var\left\{\ln\left[Sa(T_1)\right]\right\} \\ &+ 2\,\alpha\,(1-\alpha)\,\rho_{\ln\left[Sa_{avg}(T_1 ... T_N)\right],\ln\left[Sa(T_1)\right]}\,\sigma_{\ln\left[Sa_{avg}(T_1 ... T_N)\right]}\,\sigma_{\ln\left[Sa(T_1)\right]} \end{aligned} \tag{6.4}$$

The $\ln[Sa(T_i)]$ values are commonly assumed to be jointly Gaussian (Bazzurro and Cornell, 2002; Stewart et al., 2002), and for this reason the sum also is Gaussian and $Var\left\{\ln\left[Sa_{avg}(T_1 ... T_N)\right]\right\}$ and $\rho_{\ln\left[Sa_{avg}(T_1 ... T_N)\right],\ln\left[Sa(T_1)\right]}$ can be obtained through Eqn. 6.5 and Eqn. 6.6 (Baker and Cornell, 2006).

$$Var\left\{\ln\left[Sa_{avg}(T_1 ... T_N)\right]\right\} = \frac{1}{N^2}\sum_{i=1}^{N}\sum_{j=1}^{N}\left[\rho_{\ln\left[Sa(T_i)\right],\ln\left[Sa(T_j)\right]}\,\sigma_{\ln\left[Sa(T_i)\right]}\,\sigma_{\ln\left[Sa(T_j)\right]}\right] \tag{6.5}$$

$$\rho_{\ln\left[Sa_{avg}(T_1 ... T_N)\right],\ln\left[Sa(T_1)\right]} = \frac{\sum_{i=1}^{N} \rho_{\ln\left[Sa(T_i)\right],\ln\left[Sa(T_1)\right]}\,\sigma_{\ln\left[Sa(T_i)\right]}}{\sqrt{\sum_{i=1}^{N}\sum_{j=1}^{N}\left[\rho_{\ln\left[Sa(T_i)\right],\ln\left[Sa(T_j)\right]}\,\sigma_{\ln\left[Sa(T_i)\right]}\,\sigma_{\ln\left[Sa(T_j)\right]}\right]}} \tag{6.6}$$

In Equation 6.5 and Equation 6.6 $\rho_{\ln\left[Sa(T_i)\right],\ln\left[Sa(T_j)\right]}$ can be evaluated for example by using the relationship of Inoue and Cornell (Inoue and Cornell, 1990):

$$\rho_{\ln\left[Sa(T_i)\right],\ln\left[Sa(T_j)\right]} = 1 - 0.33\left|\ln\left(1/T_i\right) - \ln\left(1/T_j\right)\right| \tag{6.7}$$

Finally, if the $\ln[Sa(T_i)]$ values may be considered jointly Gaussian, Eqn. 6.3 and Eqn. 6.4 can be obtained from actual attenuation relationship, and these equations are enough to describe the complete distribution of $\ln(I_{Np})$.

7. CONCLUSIONS

A new spectral-shape-based ground motion intensity measure based on the elastic pseudo-acceleration response spectrum was proposed. I_{Np} was compared with maximum ductility, interstory drift and energy demands for single degree of freedom SDOF systems, and a steel frame subjected to seismic records with different characteristics (ordinary, near-fault pulse-like, and narrow-band motions). The parameter I_{Np} results a good predictor of the structural response of the different systems and earthquake ground motion records used, and the efficiency increases when α is between zero and one, which implies that it may have improved efficiency with respect to $Sa(T_1)$ and $Sa_{avg}(T_1 ... T_N)$. Moreover, probabilistic seismic hazard analysis for this IM can be performed by using tools currently available.

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