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A GROUND MOTION INTENSITY MEASURE TO PREDICT NON LINEAR STRUCTURAL RESPONSE AND HIGHER MODES EFFECTS

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ABSTRACT

The objective of this paper is to analyze a new ground motion intensity measure able of predicting the nonlinear behavior and higher modes effects of structures subjected to earthquakes. The new ground motion intensity measure is inspired in a proxy of the spectral shape named N_p which is directly related to the nonlinear seismic response and to the spectral shape. In fact, the parameter N_p has been successfully used for ground motion record selection strategies for the seismic performance of structures. Although the parameter N_p was proposed through the spectral shape in term of pseudo-acceleration. It will be observed that N_p can be defined through other types of spectral shapes such as those obtained with velocity, displacement, input energy, inelastic parameters and so on. Finally, it will be shown that several ground motion intensity measures are particular cases of the new generalized ground motion intensity measure here proposed which is able to predict the seismic response of building due to both nonlinear and higher modes effects.

INTRODUCTION

Currently, several parameters have been proposed with the aim of describing the ground motion potential of an earthquake (Housner, 1952; Arias, 1970; Von-Thun et al., 1988; Cosenza and Manfredi 1998; Cordova et al., 2001; Baker and Cornell, 2005; Mehanny, 2009; Bojórquez and Iervolino, 2011; Bojórquez et al., 2012). These parameters are known as ground motion intensity measures. It has been shown that among all the ground motion intensity measures, those which try to capture the spectral shape of the pseudo-acceleration spectrum are the most efficient because they have the ability to predict the structural response of buildings under earthquakes (Cordova et al., 2001; Baker and Cornell, 2005; Bojórquez and Iervolino, 2011; Bojórquez et al., 2012; Buratti 2011; 2012). Nevertheless, most of the intensity measures proposed in the literature are concentrated in the prediction of nonlinear structural response, and it is deemed necessary to take into account the higher mode effects. The aim of the present study is to define a new ground motion intensity measure able of predicting nonlinear behavior and higher modes effects of structures subjected to earthquakes. The new ground motion intensity measure is inspired in a proxy of the spectral shape named N_p which is directly related to the nonlinear seismic response (Bojórquez and Iervolino, 2011; Bojórquez et al., 2012). In fact the parameter N_p has been successfully used for ground motion record selection strategies to estimate the seismic performance of structures (Bojórquez et al., 2013). Moreover, Bojórquez and Iervolino (2011) demonstrated the potential of intensity measures based on the

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parameter N_p to predict the ductility, maximum interstory drift and hysteretic energy demands of nonlinear structures subjected to earthquake ground motion. In addition, Buratti (2011, 2012) concluded that the most efficient intensity measure is that proposed by Bojórquez and Iervolino based on the parameter N_p which was compared with several intensity measures. Further, N_p has been used to estimate the structural fragility of buildings with higher efficiency compared with other intensity measures (Bojórquez et al., 2012; Modica and Stafford, 2014). However, in spite of the great advantages of N_p, this parameter does not take into account explicitly the higher mode effects in the structural response, or it must be included by changing the initial period selected to estimate the value of N_p (Bojórquez et al., 2013). Motivated by the need to incorporate the higher mode effects in the structural response of buildings, in this paper, a new generalized ground motion intensity measure is proposed, which is denoted as I_B. This parameter is based on the parameter N_p which as it was discussed has been successfully used as proxy of the spectral shape, but in this case N_p is calculated not only with the pseudo-acceleration spectrum, also with other types of spectra. The aim of the present study is to define a new generalized ground motion intensity measure and to estimate its efficiency to predict nonlinear and higher modes effects of structures subjected to earthquakes.

TOWARD THE NEXT GENERATION OF INTENSITY MEASURES

The most used ground motion intensity measure by earthquake engineers, seismologists, and seismic design guidelines is the spectral acceleration at first mode of vibration. This parameter is very useful because is the perfect predictor of seismic response of elastic single degree of freedom systems and it is a good option for predicting the response of elastic multi degree of freedom structures dominated by the first mode of vibration. Further, some studies have demonstrated the sufficiency of $Sa(T_1)$ with respect to magnitude and distance (Shome, 1999; Iervolino and Cornell, 2005). Nevertheless, for structures dominated by higher mode effects the use of $Sa(T_1)$ could not be appropriated (Bazzurro and Cornell, 2002). Various studies have demonstrated the inefficiency of $Sa(T_1)$ for example to predict the response of buildings under near source ground motion records and narrow-band motions (Bojórquez and Iervolino, 2011). The limitations of spectral acceleration at first mode of vibration can be clearly observed with the elastic response spectra where the scatter in the spectral shape due to the effect of the elongated period, or some spectral ordinates at higher periods is not considered. Inspired by this issues Bojórquez and Iervolino (2011) have proposed the parameter N_p and the I_{Np} intensity measure, both are described in the next subsection.

I_{Np} intensity measure

To increase the efficiency of spectral acceleration, the parameter I_{Np} was proposed by Bojórquez and Iervolino (2011) which take into account the nonlinear behavior of the structures by including the parameter N_p. Although the parameter N_p was defined in a previous work by the author, here N_p and I_{Np} were again described to show the benefits or advantages of this parameter compared with the spectral acceleration intensity measure. The next definition can be found in Bojórquez and Iervolino (2011).

The parameter N_p

Recent studies suggest that the spectral shape is crucial to predict the structural response of buildings under earthquakes and for this reason the earthquake engineering and seismology community has highlighted the limitations of spectral acceleration at first mode of vibration. For example: $Sa(T_1)$ 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 nonlinear shaking, the structure may be sensitive to different spectral values associated to a range of periods, from the fundamental period until a limit value of practical interest, say T_N. To further illustrate some limitations of $Sa(T_1)$, let's consider a structure with a fundamental period T_1 equal to 1s subjected to a set of different seismic records scaled to the same $Sa(T_1)$ level.

Fig. 1 shows the response spectra of the set of records from the Mexico City case with significant local site effects. For this example, the final period is supposed to be T_N equal to 2s. It can be observed that, although the records have the same $Sa(T_1)$, the spectral ordinates are affected by significant scatter at T_N, which is likely to be reflected in the structural response. This calls for intensity measures providing information about the spectral shape in a whole region of the spectrum as $<Sa, R_{T1,T2}>$ and $Sa_{avg}(T_1,..., T_N)$.

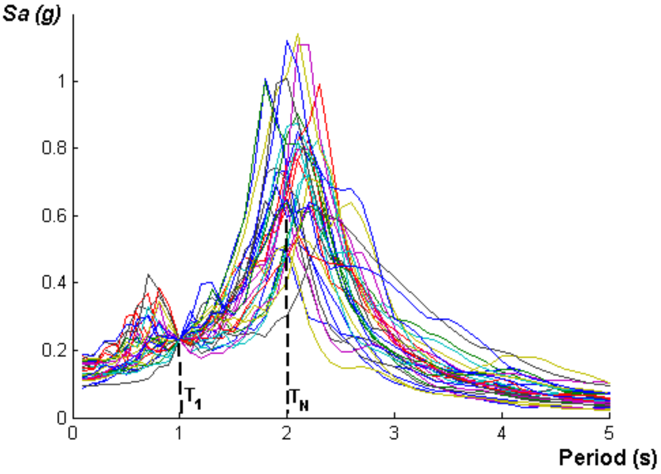

Figure 1. Response spectra for records scaled to similar $Sa(T_1)$. After Bojórquez and Iervolino (2011)

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 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 improvement may be the use of $Sa_{avg}(T_1,..., T_N)$ but normalizing it by $Sa(T_1)$. To this aim the parameter named N_p (Eq. 1) was proposed by Bojórquez and Iervolino (2011). The additional subscript *o* in N_p (see Eq. 1) is used for representing the original equation.

$$N_{po} = \frac{Sa_{avg}(T_1,..., T_N)}{Sa(T_1)} \tag{1}$$

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 period range between T_1 and T_N. For a mean N_p lower than one it is expected an average spectrum with negative slope. 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 to nonlinear structural response. Further information about this parameter can be found in Bojórquez and Iervolino (2011).

I_{Np} *ground motion intensity measure*

To incorporate the effects of nonlinear behavior in the prediction of structural response, Bojórquez and Iervolino (2011) have proposed a new scalar ground motion *IM* based on $Sa(T_1)$ and N_p which is described in the following equation:

$$I_{Np} = Sa(T_1)\, N_p^{\alpha} \tag{2}$$

In Eq. 2 the α value has to be determined. From Eq. 2, it is possible to note that 1) the spectral acceleration at first mode of vibration is a particular case of I_{Np}, and this occurs when α is equal to zero; 2) $Sa_{avg}(T_1 \ldots T_N)$ also corresponds to the particular case when $\alpha = 1$. Analyses developed by Bojórquez and Iervolino (2011) and Buratti (2011, 2012) suggest that the optimal values of α are close to 0.4, also Buratti (2011, 2012) demonstrated that this intensity measure is more efficient to predict nonlinear structural response compared with several intensity measures of the literature. Note that the previous equation provides different weights to the contributions of the spectral accelerations beyond the first-mode compared with the spectral value at T_1. Furthermore, probabilistic seismic hazard analysis can be developed using this ground motion intensity measure as Bojórquez and Iervolino (2011) have demonstrated.

The definition of I_B

Although I_{Np} has results very efficient to predict nonlinear structural response compared with other parameters (Bojórquez and Iervolino, 2011; Buratti, 2011; 2012), one of the main limitations of this intensity measure is the lack of consideration of higher mode effects, because it does not take into account spectral ordinates associated with periods lower than the fundamental periods of vibration of the structure (note that the higher mode effects can be taken into account by using a different range of spectral ordinates for the parameter N_p, as Bojórquez et al., 2013 suggest). With the aim to improve the capacity of I_{Np}, in this study, the new ground motion intensity measure I_B is proposed. This parameter is inspired in the spectral shape and it was named as I_B because it considers the prediction of structural response considering both nonlinear and higher mode effects. The new intensity measure is defined as following:

$$I_B = S(T_1)^{\alpha_1} \cdot N_p^{\alpha_2} \cdot \prod_{i=2}^{i=\#modes} \left[R_{(T1,Tmi)}^{\alpha_{3(mi)}} \right] \tag{3}$$

In Eq. 3, I_B represents the new intensity measure proposed by the first author; $S(T_1)$ represents a spectral parameter taken from any type of spectrum as in the case of acceleration, velocity, displacement, input energy, inelastic parameter and so on. N_p is similar to Eq. 1 but for different types of spectra, which can be rewritten as convenience as it is indicated in Eq. 4; $R_{(T1,Tmi)}^{\alpha_{3(mi)}}$ is defined as the ratio of a spectral parameter in the period of *mode i* of vibration of the structure ($S(T_{mi})$) and a spectral parameter at the fundamental period of vibration $S(T_1)$, where T_1 is larger than T_{mi} (see Eq. 5). Note that the subscript m is used to denote mode of vibration.

$$N_p = \frac{S_{avg}(T_1, \ldots, T_N)}{S(T_1)} \tag{4}$$

$$R_{(T1,Tmi)} = \frac{S(T_{mi})}{S(T_1)} \tag{5}$$

Eq. 3 indicates that I_B incorporate information of both nonlinear and higher mode effects in the prediction of seismic response of structures. It is important to observe that I_{Np} is a particular case of I_B when the spectral acceleration shape is selected and $\alpha_{3(mi)}$ is equal to zero for all the modes. Furthermore, parameters as spectral acceleration at first mode of vibration, spectral velocity, spectral displacement, peak ground acceleration and velocity, geometrical mean of spectral values in a range of periods, spectral input energy at first mode of vibration and others are particular cases of I_B. In addition, the parameters α_1, α_2 and α_3 must be calibrated, and $\alpha_{3(mi)}$ can change for each mode, but with the aim to express the equation in a simpler manner, α_3 can be assumed similar for all the modes under consideration, and α_1 will be equal to 1. Hence, the units of the intensity measure are similar to the units of $S(T_1)$ because N_p and R are dimensionless, then the parameter I_B can be rewritten as:

$$I_B = S(T_1)\cdot N_p^{\alpha_2}\cdot \prod_{i=2}^{i=\#modes}\left[R_{(T1,Tmi)}^{\alpha_3}\right] \tag{6}$$

A modified version of Eq. 3 which is based only on N_p is currently been developed by the first author, where it is demostrated that the best way to express the range of periods to define N_p is using the fundamental period of vibration as the initial period (Bojórquez, 2014).

SEVERAL PARTICULAR CASES OF I_B

The parameter I_B is a generalized ground motion intensity measure because several intensity measures are particular cases of this new parameter. Moreover, I_B consists of three parts, the first one is the spectral value, the second is the N_p and the last part is the R parameter. Note that several combination of I_B can be obtained if we use different spectral shape to define the three parts of I_B. For example, in the next table several particular cases of I_B are estimated taken into account three spectral parameters which are obtained from the pseudo-acceleration *Sa*, velocity *V* and displacement *D* spectra. Note that many other types of spectra can be used such as input energy, but *Sa*, *V* and *D* were selected just for illustrative purposes. Moreover, note that *D* and *Sa* are related. To better understand Table 1 it is mentioned that the case 1 was obtained by using the pseudo-acceleration spectrum to calculate the spectral value $S(T_1)$, N_p and R, and using only the two first modes of vibration. As another example the case 2 was obtained based on two modes and the pseudo-acceleration to estimate $S(T_1)$ and N_p and velocity for the third parameter R, in this case we can say that this is a hybrid case of I_B because it was obtained with two parameters. All the other cases were obtained in a similar way, note that in Table 1 (with 30 cases) only the second and few cases for the third mode were considered for illustrative purposes, which is more than enough to predict the structural response of several buildings. Further, the initial period of parameter N_p was defined equal to the fundamental period of vibration and it also can be different. Note that for the parameters *Sa*, *V* and *D* and by considering the second mode of vibration there are 27 possibilities or particular cases of I_B, which can be represented by the first 27 cases given in Table 1. Finally, the use of *Sa*, *V* and *D* in the parenthesis after I_B represents that it was obtained through these parameters, and for a particular case of I_B obtained with two modes and the same spectral parameter, the I_B can be written also as I_{BSa} (see case 1), as it will be described below.

Table 1. Particular cases of I_B estimated with spectral acceleration, velocity and displacement

Case	$S(T_1)$	N_p	$R_{(T1,Tmi)}$	**Formulation for particular cases of I_B**
1	*Sa*	*Sa*	*Sa*	$I_{B(Sa,Sa,Sa)} = Sa(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
2	*Sa*	*Sa*	*V*	$I_{B(Sa,Sa,V)} = Sa(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
3	*Sa*	*Sa*	*D*	$I_{B(Sa,Sa,D)} = Sa(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
4	*Sa*	*V*	*Sa*	$I_{B(Sa,V,Sa)} = Sa(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
5	*Sa*	*V*	*V*	$I_{B(Sa,V,V)} = Sa(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
6	*Sa*	*V*	*D*	$I_{B(Sa,V,D)} = Sa(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$

7	*Sa*	*D*	*Sa*	$I_{B(Sa,D,Sa)} = Sa(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
8	*Sa*	*D*	*V*	$I_{B(Sa,D,V)} = Sa(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
9	*Sa*	*D*	*D*	$I_{B(Sa,D,D)} = Sa(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
10	*V*	*Sa*	*Sa*	$I_{B(V,Sa,Sa)} = V(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
11	*V*	*Sa*	*V*	$I_{B(V,Sa,V)} = V(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
12	*V*	*Sa*	*D*	$I_{B(V,Sa,D)} = V(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
13	*V*	*V*	*Sa*	$I_{B(V,V,Sa)} = V(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
14	*V*	*V*	*V*	$I_{B(V,V,V)} = V(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
15	*V*	*V*	*D*	$I_{B(V,V,D)} = V(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
16	*V*	*D*	*Sa*	$I_{B(V,D,Sa)} = V(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
17	*V*	*D*	*V*	$I_{B(V,D,V)} = V(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
18	*V*	*D*	*D*	$I_{B(V,Sa,V)} = V(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
19	*D*	*Sa*	*Sa*	$I_{B(D,Sa,Sa)} = D(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
20	*D*	*Sa*	*V*	$I_{B(D,Sa,V)} = D(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
21	*D*	*Sa*	*D*	$I_{B(D,Sa,D)} = D(T_1)\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
22	*D*	*V*	*Sa*	$I_{B(D,V,Sa)} = D(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$
23	*D*	*V*	*V*	$I_{B(D,V,V)} = D(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
24	*D*	*V*	*D*	$I_{B(D,V,D)} = D(T_1)\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
25	*D*	*D*	*Sa*	$I_{B(D,D,Sa)} = D(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}$

26	*D*	*D*	*V*	$I_{B(D,D,V)} = D(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}$
27	*D*	*D*	*D*	$I_{B(D,D,D)} = D(T_1)\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}$
28 (3 modes)	*Sa*	*Sa*	*Sa*	$I_{B(Sa,Sa,Sa,Sa)} = Sa\cdot\left[\frac{Sa_{avg}(T_1,\ldots,T_N)}{Sa(T_1)}\right]^{\alpha_2}\cdot\left[\frac{Sa(T_{m2})}{Sa(T_1)}\right]^{\alpha_3}\cdot\left[\frac{Sa(T_{m3})}{Sa(T_1)}\right]^{\alpha_3}$
29 (3 modes)	*V*	*V*	*V*	$I_{B(V,V,V,V)} = V\cdot\left[\frac{V_{avg}(T_1,\ldots,T_N)}{V(T_1)}\right]^{\alpha_2}\cdot\left[\frac{V(T_{m2})}{V(T_1)}\right]^{\alpha_3}\cdot\left[\frac{V(T_{m3})}{V(T_1)}\right]^{\alpha_3}$
30 (3 modes)	*D*	*D*	*D*	$I_{B(D,D,D,D)} = D\cdot\left[\frac{D_{avg}(T_1,\ldots,T_N)}{D(T_1)}\right]^{\alpha_2}\cdot\left[\frac{D(T_{m2})}{D(T_1)}\right]^{\alpha_3}\cdot\left[\frac{D(T_{m3})}{D(T_1)}\right]^{\alpha_3}$

It is important to say that the number of combination that can be obtained for I_B depends on the number of types of spectra that are used. One of the main objectives of the second part of these study is try to find for several particular cases of I_B (all shown in Table 1) the values of α_2 and α_3 for which this intensity measure has the highest efficiency; thus the ability to predict with good accuracy the behavior of structures under earthquakes due to nonlinear and higher modes effects.

METHODOLOGY

Structural models

To estimate the efficiency of the new intensity measure I_B, six moment-resisting steel frames having 4, 6, 8, 10, 14 and 18 stories, were considered for the studies reported herein. The frames are denoted as F4, F6, F8, F10, F14 and F18, respectively. The frames, designed according to the Mexico City Seismic Design Provisions (MCSDP), have three eight-meter bays and story heights of 3.5 meters. Each frame was provided with ductile detailing and its lateral strength was established according to the MCSDP. A36 steel was used for the beams and columns of the frames. Relevant characteristics for each frame, such as the fundamental period of vibration (T_1), the period of the second mode of vibration (T_{m2}) and the seismic coefficient at yielding (C_y) are shown in Table 2 (the latter two values were established from static nonlinear analyses). An elasto-plastic model with 3% strain-hardening was used to represent the cyclic behavior. The frames were analyzed considering 3% of critical damping.

Table 2. Characteristics of the steel frame models

Frame	**Number of stories**	**Period of vibration (s)**		C_y
		T_1	T_{m2}	
F4	4	0.90	0.27	0.45
F6	6	1.07	0.34	0.42
F8	8	1.20	0.39	0.38
F10	10	1.35	0.45	0.36
F14	14	1.91	0.65	0.25
F18	18	2.53	0.87	0.185

Earthquake ground motions

The efficiency of the ground motion intensity measures is computed by considering several sets of earthquake records, which are divided in two categories. The first category having a total of 100 ground motion records obtained from the NGA database, corresponding to worldwide earthquakes, were used for the analyses of the steel frame buildings. The records used in this section were selected from earthquakes with moment magnitudes (M_w) ranging from 6.0 to 7.9, and they have been taken from sites at different epicentral distances. The selected magnitudes are representative of moderate and large earthquakes. The distribution of the records in terms of moment magnitude and distance is provided in Fig. 2. In this figure, it can be observed that the records were obtained for different distances and from different events as moment magnitudes indicate; the wide range of the selected distances and magnitudes is very important to observe the influence of these parameters on the assessment of the uncertainty of the structural response due to nonlinear and higher mode effects.

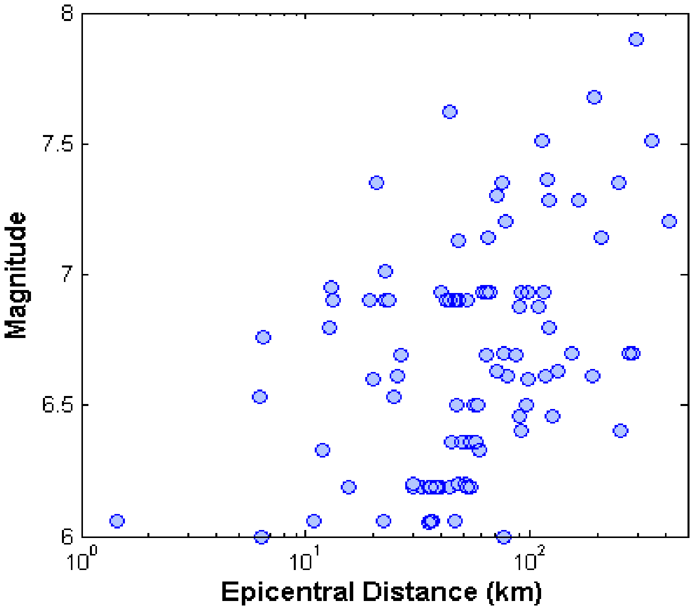

Figure. 2 Moment magnitude and epicentral distance distribution for the selected 100 records taken from the NGA database

The seismic records of the first category used in this study corresponding to all type of soil zones in accordance with the Geomatrix's Classification GMX's C3. From Soil type A corresponding to rock and soil type E to soft soil. A total of 20 records were chosen for each type of soil. The selected ground motion records were generated by different failure mechanisms: Strike-Slip, Normal, Reverse, Reverse-Oblique, Normal-Oblique, and Undefined. All the different characteristics of the selected records will show that the intensity measure I_B does not depends of the seismotectonic or soil characteristics.

In addition, the six steel frames designed according to the Mexico City Building Code are subjected to 30 soft-soil ground motions recorded in the Lake Zone of Mexico City and exhibiting a dominant period (T_s) of two seconds. Particularly, all motions were recorded in Mexico City during seismic events with magnitudes near of 7 or larger. Table 3 summarizes the main characteristics of the seismic records under consideration. In this table, while *PGA* and *PGV* stand for the peak ground acceleration and velocity, t_D is the strong-motion duration estimated according to the Trifunac and Brady criterion, which is defined as the time interval delimited by the instants of time at which the 5% and 95% of the Arias Intensity occurs. Note that the average duration of the records equals 74.4 sec.

Table 3. Earthquake ground motions

Record	Date	Magnitude	*PGA* (cm/s²)	*PGV* (cm/s)	Epicentral Distance (km)	t_D *(s)*
1	19/09/1985	8.1	178.0	59.5	366	34.8
2	21/09/1985	7.6	48.7	14.6	323	39.9
3	25/04/1989	6.9	45.0	15.6	293	37.8
4	25/04/1989	6.9	68.0	21.5	294	65.5
5	25/04/1989	6.9	44.9	12.8	289	65.8
6	25/04/1989	6.9	45.1	15.3	286	79.4
7	25/04/1989	6.9	52.9	17.3	295	56.6
8	25/04/1989	6.9	49.5	17.3	293	50.0
9	14/09/1995	7.3	39.3	12.2	303	53.7
10	14/09/1995	7.3	39.1	10.6	303	86.8
11	14/09/1995	7.3	30.1	9.62	286	60.0
12	14/09/1995	7.3	33.5	9.37	298	77.8
13	14/09/1995	7.3	34.3	12.5	295	101.2
14	14/09/1995	7.3	27.5	7.8	304	85.9
15	14/09/1995	7.3	27.2	7.4	303	68.3
16	09/10/1995	7.5	14.4	4.6	536	85.5
17	09/10/1995	7.5	15.8	5.1	537	97.6
18	09/10/1995	7.5	15.7	4.8	537	82.6
19	09/10/1995	7.5	24.9	8.6	537	105.1
20	09/10/1995	7.5	17.6	6.3	537	104.5
21	09/10/1995	7.5	19.2	7.9	539	137.5
22	09/10/1995	7.5	13.7	5.3	540	98.4
23	09/10/1995	7.5	17.9	7.18	541	62.3
24	11/01/1997	6.9	16.2	5.9	379	61.1
25	11/01/1997	6.9	16.3	5.5	379	85.7
26	11/01/1997	6.9	18.7	6.9	381	57.0
27	11/01/1997	6.9	22.2	8.6	381	76.7
28	11/01/1997	6.9	21.0	7.76	380	74.1
29	11/01/1997	6.9	20.4	7.1	380	81.6
30	11/01/1997	6.9	16.0	7.2	383	57.5

The different sets of ground motion records have been considered to represent all the types of soils, in such a way that the earthquake motions are representative of different spectral shapes. This let to show the potential of I_B to predict the seismic behavior of framed buildings subjected to ground motion records with very different characteristics and considering different seismic sources.

Numerical results

To assess the efficiency of the ground motion intensity measures, all the records have been scaled for different values of spectral acceleration at first mode of vibration in a range from 0.5g until 1.2g to perform incremental dynamic analysis (Vamvatsikos and Cornell, 2002). For the dynamic analyses the well-known computer program RUAUMOKO (Carr, 2008) was used. Scaling levels less than 0.5g have not been selected since in this range of intensities the nonlinear behavior is despicable in the steel structures models. From the dynamic analyses, the maximum interstory drift for each structure and type of soil were obtained. The maximum interstory drift was selected since it is the main parameter used in earthquake engineering for structural performance assessment, and it is the key engineering demand parameter used by the seismic design codes.

The efficiency is calculated by means of the standard deviation of the natural logarithm of the maximum interstory drift $\sigma_{\ln(\gamma)}$ (note that γ was used to represent the maximum interstory drift) in the range of intensity levels given previously. First, linear regression analysis was used to calculate the interstory drift in terms of intensity (expected interstory drift) and then the standard deviation of the natural logarithm was computed as the difference with the expected and the real drift. The most efficient intensity measure is that able to minimize the standard deviation of the structural demand, hence the reduction of the uncertainty in the seismic response. The efficiency of I_B is compared with I_{Np} and $Sa(T_1)$. Note that I_B is computed for several values of α_2 and α_3, in particular when α_3 is equal to zero I_{Np} is a particular case of I_B as Table 1 case 1 suggests. The same occurs when α_2 and α_3 are equal to zero $Sa(T_1)$ is also a particular case of I_B (this is valid when the case 1 of I_B is selected), in such a way that in this study it is only necessary to estimate the efficiency of I_B. In fact, the main objective of this study is to find for the particular cases of I_B illustrated in Table 1, the values of α_2 and α_3 for which the I_B is the most efficient. Thus, the study is focus into propose the best alternative form to express I_B to increase its prediction of the seismic response of building due to nonlinear behavior and higher modes effects. Finally, it is important to say that the efficiency was computed for the steel structures subjected to the different types of soils separately; however, for the sake of brevity only the results of the standard deviation when all the records were selected simultaneously to compute the efficiency were included in the present study.

RESULTS FOR I_{Np}

For the parameter I_{Np}, α values from zero to one were used to verify the value that increases the efficiency of this parameter as predictor of the structural response. Because I_{Np} is a particular case of I_B when the pseudo-acceleration spectrum is selected, and for α_2 equals to one and α_3 equal to zero. It was not necessary to estimate the relation between I_{Np} and the structural demands, only the relation of I_B with structural response was computed. In fact, it is not necessary to estimate $\sigma_{\ln(\gamma)}$ for $Sa(T_1)$ or for I_{Np} since they are parameters that can be obtained with I_B. As Bojórquez and Iervolino (2011) suggest, in this study (see below) the value of α that minimize the standard deviation of the structural response was also 0.4. For this reason, I_{Np} can be estimated as it is indicated in the following equation.

$$I_{Np} = Sa(T_1) \cdot N_p^{0.4} \tag{7}$$

RESULTS FOR I_B

For the analyses of the selected study cases of I_B only the second and third mode of vibration have been considered (see Table 1). It is important to say that the explicit consideration of two modes of vibration is more than enough to predict the seismic response of most of the framed buildings. However, in this work the third mode was also included with the aim to show that in the steel frames under consideration is not necessary the use of more than two mode of vibrations. Fig. 3 illustrates the influence of the parameters α_2 and α_3 to calculated the standard deviation of the maximum interstory drift for I_B using the case 1 (I_B defined with the pseudo-acceleration spectral shape). In addition, the surfaces of Figure 3 were plotted for different valued of T_N (this is a key parameter to compute N_p). As it was discussed previously and Bojorquez and Iervolino suggest a value of α=0.4 let the parameter I_{Np} have the highest efficiency as Intensity measure especially for $T_N=2*T_1$. It can be observed in Fig. 3 which indicates that a value of α_2=0.4 when α_3=0 reduced the standard deviation or the uncertainty in the estimation of the structural response in terms of maximum interstory drift. Note that a value of α_3 equal to 0.2 is proposed as can be observed in the surfaces this value represents a minimum in Figure 3 for all the T_N values analyzed. Based on the results obtained in the present study, it can be concluded that the values of α_1, α_2 and α_3 that reduce the standard deviation for case 1 of I_B are 1, 0.4 and 0.2 respectively. For this case, Eq. 3 can be rewritten as it is indicated in Eq. 8. Note that Eq. 8 has included the subscript Sa in I_B, since the three parts of the intensity measures depend of the pseudo-acceleration spectrum.

$$I_{BSa} = Sa(T_1) \cdot N_p^{0.4} \cdot R_{T1,T2}^{0.2} \tag{8}$$

Note that in the figures when α_2 y α_3 are equals to zero the parameter $Sa(T_1)$ is obtained. For this case, it is observed that $\sigma_{\ln(\gamma)}$ is larger for $Sa(T_1)$ comparing with the new intensity measures I_{Np} and I_B, which are better related with the structural response. Further, due to the dimensionless of N_p and $R_{T1,T2}$, the intensity I_B will have the units of the selected parameter in the first terms. It is important to emphasize that similar results were obtained for all the frames and considering the 30 cases of Table 1; nevertheless due to the large number of figures only the results of Frame F10 were included in the present work for illustrative purposes.

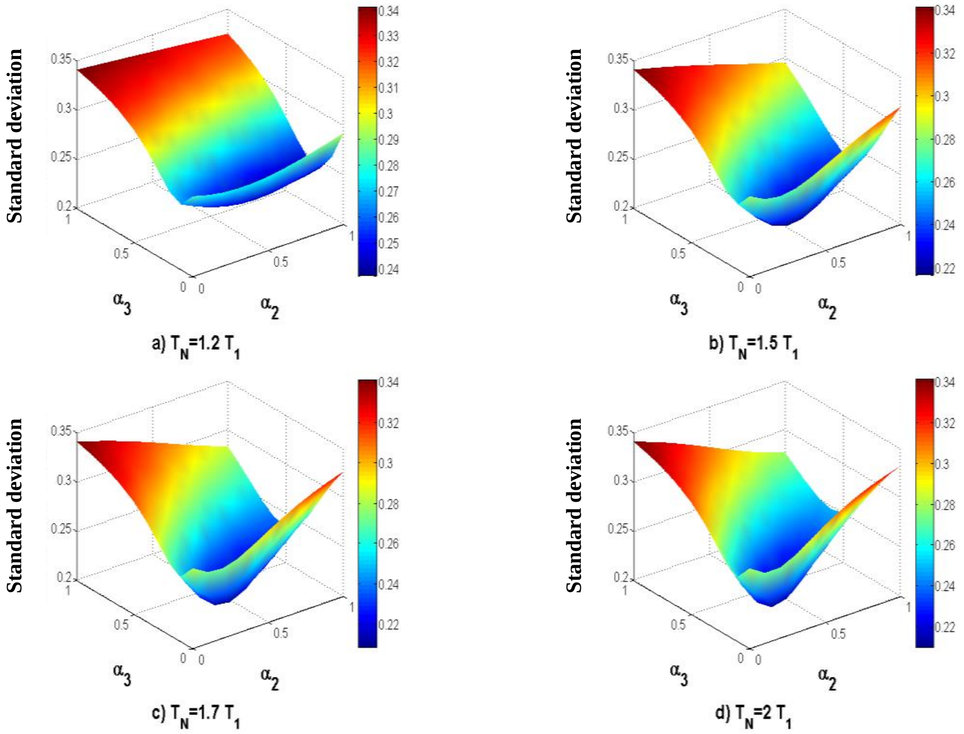

Figure 3. $\sigma_{\ln(\gamma)}$ for Frame F10 subjected to all the earthquake ground motions (I_B estimated with case 1)

Finally, with the aim to obtain the results for all the cases shown in Table 1 for I_B, the same procedure described before (the computation of the standard deviation surfaces) was used. Table 4 shows the three cases with the highest efficiency in the prediction of the maximum interstory drift with the solution obtained for the alpha parameters. It is observed that the alphas are similar for the three cases with most efficiency, and only two modes were necessary. Although the results included in the present paper are very brief, currently an extended version of the paper is being prepared by the authors.

Table 4. Particular cases of I_B with the highest efficiency.

Case	Formulation
1	$I_B = Sa(T_1)\cdot\left[\frac{Sa_{avg}(T_1\ldots T_N)}{Sa(T_1)}\right]^{0.4}\cdot\left[\frac{Sa(T_{modo2})}{Sa(T_1)}\right]^{0.2}$
4	$I_B = Sa(T_1)\cdot\left[\frac{Vel_{avg}(T_1\ldots T_N)}{Vel(T_1)}\right]^{0.4}\cdot\left[\frac{Sa(T_{modo2})}{Sa(T_1)}\right]^{0.2}$
13	$I_B = Vel(T_1)\cdot\left[\frac{Vel_{avg}(T_1\ldots T_N)}{Vel(T_1)}\right]^{0.4}\cdot\left[\frac{Sa(T_{modo2})}{Sa(T_1)}\right]^{0.2}$

CONCLUSIONS

The efficiency of $Sa(T_1)$, I_{Np} and several particular cases of the new proposed ground motion intensity measure I_B to predict the structural response due to nonlinear behavior and higher mode effects was estimated. For this aim, six moment resisting steel frames were subjected to various earthquake records obtained from different types of soils and seismic sources. The results preliminary suggests that I_B is more efficient than the traditional spectral acceleration at first mode of vibration, and I_{Np} (which is a very good predictor of the structural response in terms of nonlinear behavior). As in the case of the study developed by Bojórquez and Iervolino this work also suggests a value of α = 0.4 for I_{Np}. In the case of I_B which was calculated with several spectral shape parameters, the study suggests that I_B is very efficient and promising when it is obtained using only pseudo-acceleration spectral shape. The study also shows that it is only necessary the use of two modes (at least for the structures under consideration) to define the new intensity measure and to get high efficiency. Finally, a value of α_1 = 1, α_2 = 0.4 y α_3 = 0.2 is preliminary proposed to define the new ground motion intensity measure I_B.

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REFERENCES

Arias A (1970) “A measure of earthquake intensity”, *Seismic Design for Nuclear Power Plants, edited* by R. J. Hansen, MIT Press, Cambridge, MA, 438-483

Baker JW and Cornell CA (2005) “A vector-valued ground motion intensity measure consisting of spectral acceleration and epsilon”, *Earthquake Engineering and Structural Dynamics*, 34:1193-1217

Bazzurro P and Cornell CA (2002) “Vector-valued probabilistic seismic hazard analysis (VPSHA)”, *Proceedings of the Seventh U.S. National Conference on Earthquake Engineering*, Boston, MA.

Bojórquez E and Iervolino I (2011) “Spectral shape proxies and nonlinear structural response”. *Soil Dynamics and Earthquake Engineering*, 31(7):996-1008

Bojórquez E, Iervolino I, Reyes-Salazar A and Ruiz SE (2012) “Comparing vector-valued intensity measures for fragility analysis of steel frames in the case of narrow-band ground motions”, *Engineering Structures*, 45:472–480

Bojórquez E, Bojórquez J, Ruiz SE and Reyes-Salazar A (2013) “Prediction of inelastic response spectra using artificial neural networks”, *Mathematical Problems in Engineering*.

Buratti N (2011) “Confronto tra le performance di diverse misure di intensità dello scuotimento sísmico”, *National Congress of Engineering in Italy*, ANIDIS, Bari Italy (In Italian)

Buratti N (2012) “A comparison of the performances of various ground–motion intensity measures”, *The 15th World Conference on Earthquake Engineering, Lisbon, Portugal*, 24-28 September

Carr A (2008) RUAUMOKO inelastic dynamic analysis program, Department of Civil Engineering, University of Cantenbury, New Zealand.

Cordova PP, Dierlein GG, Mehanny SSF, Cornell CA (2001) “Development of a two parameter seismic intensity measure and probabilistic assessment procedure”, *The second U.S.-Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinforce Concrete Building Structures, Sapporo, Hokkaido*, 187-206

Cosenza E and Manfredi G, (1998) “A seismic design method including damage effect”, *11th European Conference on Earthquake Engineering,* Paris, France

Housner GW (1952) “Spectrum intensities of strong motion earthquakes”, *In: Proceedings of the symposium on earthquake and blast effects on structures. Earthquake Engineering Research Institute 1952*

Iervolino I and Cornell CA (2005) “Records selection for nonlinear seismic analysis of structures”, *Earthquake Spectra*, 21:685-713

Mehanny SSF (2009) “A broad-range power-law form scalar-based seismic intensity measure”, *Engineering Structures,* 31: 1354-1368

Modica A and Stafford P (2014) “Vector fragility surfaces for reinforced concrete frames in Europe”, *Bulletin of Earthquake Engineering*, 2014

Shome N (1999) Probabilistic seismic demand analysis of nonlinear structures, Ph.D. Thesis, Department of Civil and Environmental Engineering, Stanford University

Vamvatsikos D and Cornell CA (2002) “Incremental dynamic analysis”, *Earthquake Engineering and Structural Dynamics*, 31:491-514

Von-Thun JL, Rochin LH, Scott GA, Wilson, JA (1988) “Earthquake ground motions for design and analysis of dams”, *In: Earthquake Engineering and Soil Dynamics: II. Recent advance in ground-motion evaluation. Geotechnical Special Publication 20,* ASCE. New York, 463–481