EFFICIENCY ASSESSMENT OF SCALAR INTENSITY MEASURES IN PREDICTING ENGINEERING DEMAND PARAMETERS

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ABSTRACT

How to select strong ground motion records (SGMRs) as the input of the Nonlinear Time History Analysis (NLTHA) is an important challenge because of its significant influence on the response analysis. In the probabilistic based earthquake engineering, the seismic demand will be most valuable, if the dispersion around the mean is reduced, so finding methods to selection and/or scaling of the SGMRs which can reduce the response dispersion is very important. Strong ground motion intensity measures (IMs) are important parameters which can play an important role in the selection and scaling of SGMRs. The main purpose of this paper is to show how the application of different IMs affects the results of nonlinear time history analysis. For this purpose, a 3-story 3-D steel moment frame subjected to a set of 66 SGMRs. The results illustrate that, the sensitivity of the results of NLTHA to the selected IMs are significantly different, So use of the appropriate parameters in the selection and scaling procedure can reduce the dispersion of NLTHA.

INTRODUCTION

The use of Non-Linear Dynamic Analyses ensues significant uncertainties on the seismic demand, especially when real SGMRs are used. Porter et al. (2002) showed that among all sources of uncertainty such as; material properties, design assumptions and earthquake-induced ground motion the latter seems to be the most unpredictable and variable. Due to the unpredictable nature of earthquake ground motions, to achieve a reliable result, we should either use a large set of recorded ground motions, which can be time consuming, or find a new method to achieve the results with the same level of reliability, but with reduced number of records. Several methods have been proposed to select and scale records using one or more IMs, since SGMR intensity measures (IMs) are significant parameters affecting the results of nonlinear time history analysis.

Shome et al. (1998a) showed that halving the dispersion in ground-motion intensities decreases the necessary number of NLTHA by a factor of 4 keeping the same level of dispersion in estimated engineering demand parameters (EDPs). Baker and Cornell (2006) proposed $\varepsilon_{SA}(T_1)$ as a new IM that can be an indicator of spectral shape, which is an important effective parameter in NLTHA. Mousavi et al. (2011) showed that the correlation between $\varepsilon_{SA}(T_1)$ and nonlinear time history responses is strong enough to use ε -filtration for the selection of SGMRs with least bias in the prediction of the structural nonlinear responses.

Due to the fact that capability of different IMs in predicting nonlinear responses are not equal and relates to the correlations between these parameters and desired EDPs; in case of a specific structure, efficiency assessment of scalar IMs is investigated in this paper. For this purpose, NLTHA performed for a 3-story 3-D steel moment frame subjected to a set of 33 pairs of horizontal SGMRs. Scaling methods are: PGA based, SA(T1) based and the conventional code based approach.

STRUCTURAL MODEL, GROUND MOTIONS AND ENGINEERING DEMAND PARAMETERS (EDPs)

The structural model used in this study is a 3-story 3-D steel moment frame that designed considering of weak beam-strong column theory with the concentrated plastic hinges assigned at the beam ends and bottom of the first story columns. The first three horizontal x-direction periods are: 1.04, 0.34 and 0.19 sec. For the nonlinear time history analyses, 5% Rayleigh damping is assigned to the first mode and the third mode at which the cumulative mass participation exceeds 95%, according to this, the second mode damping ratio should be considered 4.45%. It should be noted that this study does not address the structural variability effects on the IMs efficiency.

A set of 33 pairs of horizontal SGMRs has selected from general far-field ground motion set developed as a generalization of SGMs proposed by Haselton (2009). The earthquake moment magnitudes for the selected records, which are relatively large, ranged from 6.5 to 7.6, and the distance from source to site of them are greater than 10 km (average of Joyner-Boore and Campbell distances). This general set and further information are available at

http://www.csuchico.edu/structural/researchdatabases/ground_motion_sets.shtml

The main EDP which was considered in this paper is Maximum Inter-story Drift Ratio (MIDR) over building height; as well as the maximum base shear and moment, the conclusions are based on these EDPs, and it may be different for some other structural responses (e.g., peak floor accelerations or element force demands). The focus on MIDR was selected because it is a parameter of great interest for both code-based design checks as well as performance-based engineering and there is much research experience in predicting this parameter.

INVESTIGATED IMSs

The IMs investigated in this study consist of two groups. The first group, which are listed in Table 5, considers any important structural characteristics which affect the structural nonlinear responses. When a vibration property of the structure is involved, the improved methods of scaling ground motions can be obtained. Scaling records to a target value of the first mode elastic spectral acceleration, $Sa(T_1)$, from the code-based design spectrum or PSHA-based uniform hazard spectrum at the fundamental vibration period of the structure, T_1, provides improved results for structures whose response is dominated by their first-mode (Shome et al.,1998b). However, this scaling method becomes less accurate and efficient for structures responding significantly in their higher vibration modes or far into the inelastic range (Alavi and Krawinkler, 2000). So the second group of IMs which are investigated in this study are structural specific IMs. These IMs listed in Table 6. It should be noted that both structural dependent and independent IMs are more than IMs listed in Table 5 and 6, but in this paper only well-known intensities are considered.

EFFICIENCY OF SCALAR IMs IN PREDICTIONG EDPs

As mentioned previously, capability of different IMs in predicting nonlinear responses are not equal and relates to the correlations between these parameters and desired EDPs in case of a specific structure. Fig. 1 to 3 show the regression between natural logarithm of 3 EDPs and IMs investigated in this study. The coefficient of determination, R^2, is an index to show how well data fit to the linear statistical model. According to these figures, influence of the different IMs on the nonlinear responses of a structure can be dramatically different. Furthermore, this influence is not uniform for different EDPs, for example, R^2 value between Housner intensity and MIDR is 0.73, while it is 0.64 between Housner intensity and max base shear. It is therefore reasonable to seek for IMs which are more suitable for the prediction of intended EDP.

Table 1 presents the correlation coefficient between IMs and various EDPs, as described in previous sections. This parameter is a single number that describes the degree of relationship between two variables. According to Table 1, the highest correlation between IMs and nonlinear responses can be observed between velocity response intensity and MIDR (with correlation coefficient value of 0.89). It should be noted that the correlation between velocity response intensity and maximum base shear and base moment are different from fore mentioned value for MIDR, with correlation coefficient value of 0.83 and 0.85 respectively. Another important conclusion that can be concluded from Table 1, is the weak correlation between peak ground acceleration (PGA) and nonlinear responses (with a range of 0.2 to 0.38), what is the opposite to one’s expectation; However it is visible using of average spectral acceleration is a better representative of nonlinear responses, from one perspective, this could prove the importance of spectrum shape on the NLTHA outputs. $Sa(T_1)$, shows high correlations (with a range of 0.79 to 0.89), this is because of the high participation of the first mode in the total responses of the first mode dominant structure.

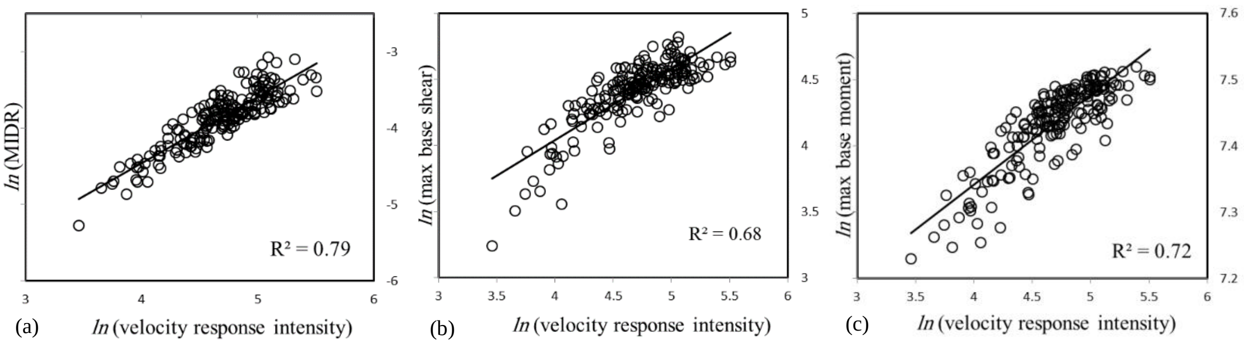

Figure1. Regression between the natural logarithm of velocity response intensity vs. the: (a) ln (MIDR) (b) ln (Max base shear) (c) ln (Max base moment)

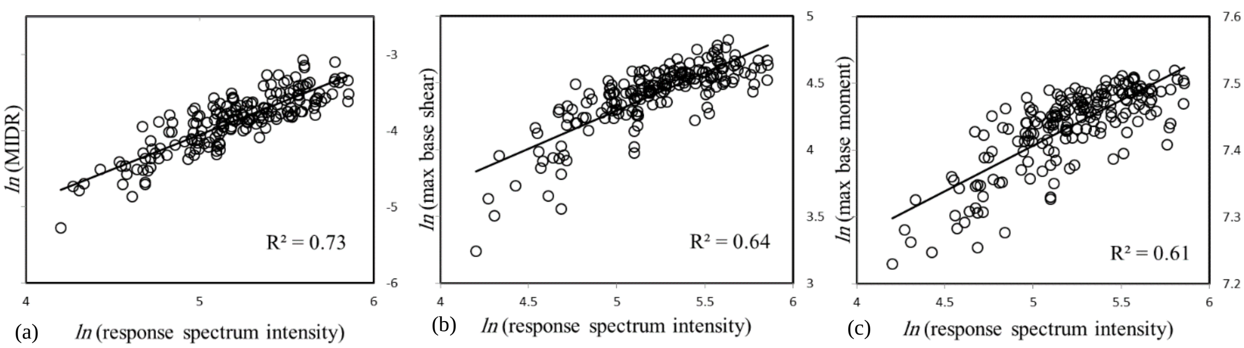

Figure 2. Regression between the natural logarithm of response spectrum intensity [Housner] vs. the: (a) ln (MIDR) (b) ln (Max base shear) (c) ln (Max base moment)

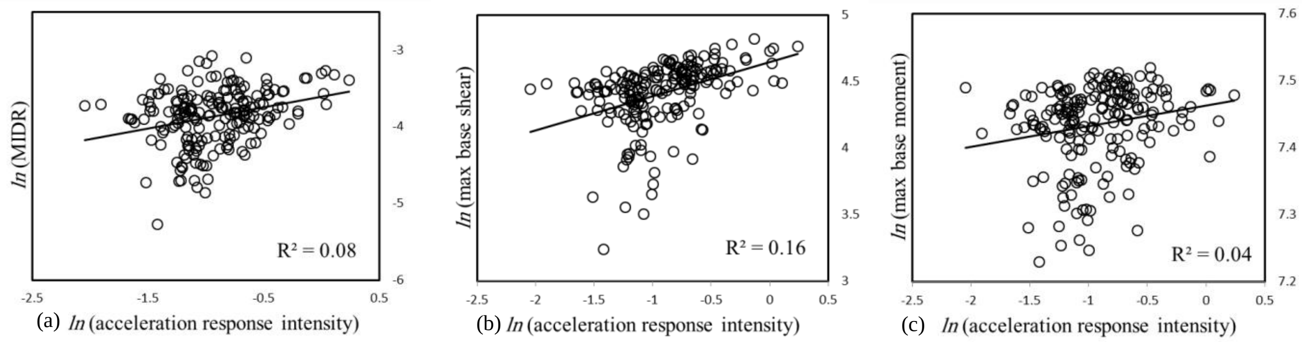

Figure 3. Regression between the natural logarithm of acceleration response intensity vs. the: (a) ln (MIDR) (b) ln (Max base shear) (c) ln (Max base moment)moment

Table 1. Correlation coefficient between natural logarithm of IMs and EDPs investigated in this paper

IM No.	Intensity measure	correlation coefficient		
		MIDR	Max Base Shear [ton]	Max Base Moment [ton.m]
1	Peak Ground Acceleration	0.33	0.38	0.20
2	Peak Ground Velocity	0.75	0.70	0.67
3	Peak Ground Displacement	0.55	0.50	0.54
4	Root Mean Square Acceleration	0.38	0.49	0.34
5	Root Mean Square Velocity	0.66	0.65	0.64
6	Root Mean Square Displacement	0.48	0.46	0.49
7	Arias Intensity	0.52	0.58	0.48
8	Velocity Intensity	0.57	0.51	0.59
9	Displacement Intensity	0.44	0.40	0.46
10	Characteristic Intensity	0.48	0.57	0.44
11	Specific Energy Density	0.69	0.63	0.67
12	Cumulative Absolute Velocity	0.54	0.56	0.54
13	Cumulative Absolute Displacement	0.60	0.53	0.60
14	Cumulative Absolute Impulse	0.48	0.43	0.49
15	Acceleration Response Intensity	0.29	0.40	0.20
16	Velocity Response Intensity	0.89	0.83	0.85
17	Displacement Response Intensity	0.55	0.49	0.55
18	Housner Intensity	0.84	0.78	0.81
19	Response Spectrum Intensity	0.85	0.80	0.78
20	Frequency Ratio1	0.49	0.39	0.52
21	Frequency Ratio2	0.22	0.18	0.28
22	Effective Peak Acceleration	0.29	0.40	0.20
23	Efective Peak Velocity	0.89	0.83	0.85
24	Effective Peak Displacement	0.55	0.49	0.54
25	Average Spectral Acceleration	0.78	0.74	0.71
26	Average Spectral Velocity	0.77	0.71	0.71
27	Average Spectral Displacement	0.58	0.53	0.58
28	A95 Parameter [Sarma & Yang, 1987]	0.33	0.38	0.20
29	Medium Period IM	0.75	0.68	0.70
30	Predominant Period (Tp)	0.31	0.11	0.16
31	Mean Period(Tm)	0.51	0.31	0.51
32	Bracketed duration	0.33	0.27	0.37
33	Uniform Duration	0.32	0.28	0.41
34	Significant Duration	0.27	0.18	0.35
35	SA(T1)	0.89	0.79	0.86
36	SA(T2)	0.36	0.49	0.29
37	SA(T1)/SA(T2)	0.44	0.25	0.47

TARGET LEVEL OF IMs PREDICTION

As mentioned in the previous section, some of the IMs have a strong correlation with the nonlinear responses, but the choice of the target level for most of them is a practical challenge in the record selection and scaling procedure. One approach is to establish a ground-motion prediction model that predicts the probability distributions of a specific IM for a specified earthquake event. Another approach is based on use of IMs which have a ground-motion prediction model like PGA, PGV or Sa(T_1) and derivation of the target level from the regression analysis between them and desired IMs. The correlation between predictable and unpredictable IMs is studied in this section. Regression analysis is used to develop an analytical equation for the evaluation of target IMs. Fig. 4 shows velocity response intensity VS. PGA, PGV and Sa(T_1). As it can be seen the velocity response intensity is enough correlated with PGV and so this regression model can be used to predict the target value.

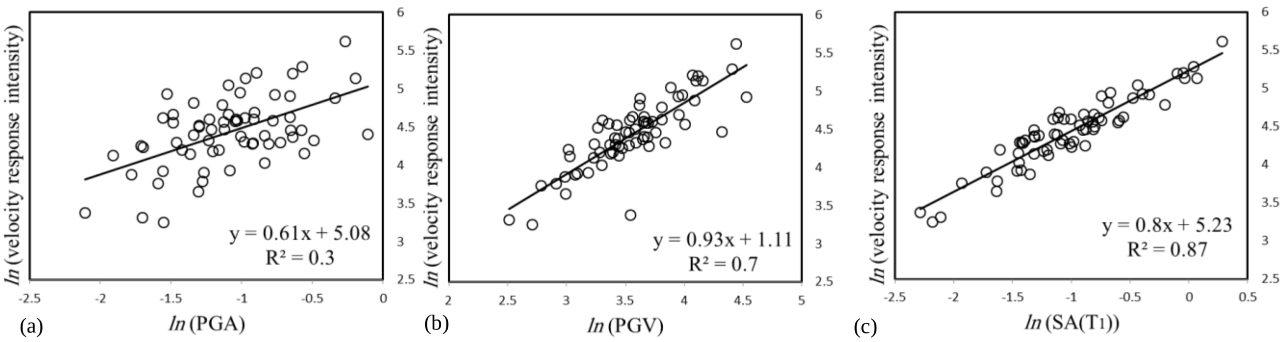

Figure 4. Regression between the natural logarithm of velocity response intensity vs. the a) ln (PGA), b) ln (PGV), c) ln (Sa(T_1))

ASSESSMENT of SCALLING METHODS

Efficiency of the three types of scaling methods used in this study, i.e., PGA, SA (T_1) and the code based methods, are investigated with respect to the structural responses. Coefficient of Variation, (C.O.V), is shown in Table 2 to 4 for 3 EDPs for each scaling methods. This parameter shows the extent of variability in the relation to mean of the population. It can be seen that, mean value of the EDPs for Sa(T1) and code based methods are close together, but standard deviation in the code based is higher. On the other hand, it seems scaling ground motions by PGA results has bias as compare with other methods with high dispersion it has. It can be concluded that the use of dynamic property of a structure that ground motions are selected for, can lead to the results with the less dispersion around the mean.

According to Table 2 to 4, the coefficients of variation for three EDPs are completely different. As it can be seen that MIDR and maximum base moment has a maximum and minimum dispersion in the all three scaling methods, respectively. Based on these observations, the efficiency of a scaling method should be assessed according to the type of desired EDP.

Table 2. Standard deviation, mean and coefficient of variation of PGA scaling method

EDPs	Standard deviation	Mean	C.O.V
Max Base Shear (ton)	17.76	73.14	0.24
Max Base Moment (ton.m)	118.16	1646.59	0.07
MIDR	0.007573	0.01846	0.41

Table 3.Standard deviation, mean and coefficient of variation of Sa(T1) scaling method

EDPs	Standard deviation	Mean	C.O.V
Max Base Shear (ton)	11.70	92.35	0.126
Max Base Moment (ton.m)	46.25	1734.59	0.026
MIDR	0.005016	0.02485	0.201

Table 4.Sandard deviation, mean and coefficient of variation of Code based scaling method

EDPs	Standard deviation	Mean	C.O.V
Max Base Shear (ton)	15.70	90.2	0.17
Max Base Moment (ton.m)	193.02	1744.04	0.11
MIDR	0.009921	0.02579	0.38

CONCLUSION

The IMs effects on the results of the nonlinear time history analysis have been investigated in this paper. For this purpose, a 3-story 3-D steel moment frame subjected to a set of 66 SGMRs. The results showed that: 1) Velocity response intensity, as a predictor of non-linear responses, is an efficient parameter that can be used to reduce bias in the structural non-linear responses. The correlation between this IM and the non-linear response is about 66% better than the correlation between PGA and the responses, 2) Spectral shape can play a significant role in the final results, 3) Based on the results of regression analysis, an analytical equation can be proposed to predict of the target IMs based on a given PGA, PGV or SA(T1), 4) Scaling ground motions by PGA results in biased EDPs compared with SA (T1) and the conventional code based approach and 5) When a vibration property of the structure is involved, the improved methods of scaling ground motions can be obtained.

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APPENDIX A

Table 5. Structural Independent Intensity measures

IM No.	IM Name	Definition	Units
1	Peak ground acceleration	$PGA = max\lvert\ddot{u}_g\rvert$	g
2	Peak ground velocity	$PGV = max\lvert\dot{u}_g\rvert$	cm/s
3	Peak ground displacement	$PGD = max\lvert u_g\rvert$	cm
4	Root mean square acceleration	$A_{rms} = \sqrt{\frac{1}{D}\int_0^D [\ddot{u}_g(t)]^2 dt}$	g
5	Root mean square velocity	$V_{rms} = \sqrt{\frac{1}{D}\int_0^D [\dot{u}_g(t)]^2 dt}$	cm/s
6	Root mean square displacement	$D_{rms} = \sqrt{\frac{1}{D}\int_0^D [u_g(t)]^2 dt}$	cm
7	Arias intensity	$I_A = \frac{\pi}{2g}\int_0^D [\ddot{u}_g(t)]^2 dt$	cm/s
8	Velocity intensity	$I_V = \frac{1}{PGV}\int_0^D [\dot{u}_g(t)]^2 dt$	cm
9	Displacement intensity	$I_D = \frac{1}{PGD}\int_0^D [u_g(t)]^2 dt$	cm.s
10	Characteristic intensity	$I_C = A_{rms}{}^{1.5}\, T_D{}^{0.5}$ *	$m^{1.5}/S^{2.5}$
11	Specific energy density	$SED = \int_0^D [\dot{u}_g(t)]^2 dt$	cm^2/s
12	Cumulative absolute velocity	$CAV = \int_0^D \lvert\ddot{u}_g(t)\rvert dt$	cm/s
13	Cumulative absolute displacement	$CAD = \int_0^D \lvert\dot{u}_g(t)\rvert dt$	cm
14	Cumulative absolute impulse	$CAI = \int_0^D \lvert u_g(t)\rvert dt$	cm.s
15	Acceleration response intensity	$ASI = \int_{0.1}^{0.5} Sa(T,\xi = 0.05)dT$	g.s
16	Velocity response intensity	$VSI = \int_{0.7}^{2.0} Sv(T,\xi = 0.05)dT$	cm
17	Displacement response intensity	$DSI = \int_{2.5}^{4.0} Sd(T,\xi = 0.05)dT$	cm.s
18	Housner intensity	$HI = \int_{0.1}^{2.5} PSV(T,\xi = 0.05)dT$	cm
19	Response spectrum intensity [Housner]	$SI = \int_{0.1}^{2.5} Sv(T,\xi = 0.05)dT$	cm
20	Frequency ratio 1	$FR_1 = PGV/PGA$	s
21	Frequency ratio 2	$FR_2 = PGD/PGV$	s
22	Effective peak acceleration	$EPA = \frac{Sa_{avg}(T,\xi)\big\rvert_{0.1}^{0.5}}{2.5}$	g
23	Effective peak velocity	$EPV = \frac{Sv_{avg}(T,\xi)\big\rvert_{0.7}^{2.0}}{2.5}$	cm/s
24	Effective peak displacement	$EPD = \frac{Sd_{avg}(T,\xi)\big\rvert_{2.5}^{4.5}}{2.5}$	cm
25	Average spectral acceleration	$Sa_{avg} = \sum_{T_1}^{T_n} \frac{Sa(Ti,\xi)}{n}$	g
26	Average spectral velocity	$Sv_{avg} = \sum_{T_1}^{T_n} \frac{Sv(Ti,\xi)}{n}$	cm/s

27	Average spectral displacement	$$Sd_{avg} = \sum_{T_1}^{T_n} \frac{Sd(Ti, \xi)}{n}$$	cm
28	A95 parameter [Sarma and Yang,1987]	The acceleration level below which 95% of the total Arias intensity is contained	g
29	Medium period IM	$I = PGV(T_D{}^{0.25})$ *	s
30	Predominant period (Tp)	The period at which the maximum spectral acceleration occurs in an acceleration response spectrum calculated at 5% damping	s
31	Mean period (Tm)	$$T_m = \frac{\sum C_i^2 / f_i}{\sum C_i^2}$$ Ci: are the Fourier amplitudes fi: represent the discrete Fourier transform frequencies between 0.25 and 20 Hz.	s
32	Bracketed duration	The total time elapsed between the first and the last excursions of a specified level of acceleration (default is 5% of PGA)	s
33	Uniform duration	The total time during which the acceleration is larger than a given threshold value (default is 5% of PGA)	s
34	Significant duration	The interval of time over which a proportion (percentage) of the total Arias Intensity is accumulated (default is the interval between the 5% and 95% thresholds)	s

* $T_D = t(0.95I_A) - t(0.05I_A)$

Table 6. Structural dependent Intensity measures

IM No.	IM	Definition	units
35	Sa(T1)	elastic spectral acceleration at the fundamental vibration period of the structure	g
36	Sa(T2)	elastic spectral acceleration at the second vibration period of the structure	cm/s
37	Sa(T1)/Sa(T2)	Ratio of elastic spectral acceleration at the first vibration period of the structure to the second vibration period	cm