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Ground motion intensity measures for performance-based earthquake engineering

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ABSTRACT: A critical issue in the probabilistic framework for performance-based earthquake engineering is the choice of ground motion intensity measure(s). In an effort to identify optimum intensity measure(s), a comprehensive parametric statistical study based on 881 earthquake record components was conducted to identify the correlation between (1) seismological variables (SV's) and traditional ground motion intensity measures (IM's), and (2) various nonlinear SDOF response parameters indicative of damage. A set of new nonlinear SDOF-based ground motion intensity measures is defined and proposed to complement the elastic spectral acceleration at the initial fundamental period (T_0) of the structure. The correlation performance of the new intensity measures is compared to that of traditional intensity measures. The new intensity measures are shown to be efficient in reducing the dispersion of nonlinear single-degree-of-freedom (SDOF) response parameters across a wide range of initial periods T_0 and strength levels. A few alternative vectors of optimum ground motion intensity measures are proposed, which consist of the 5 percent damped elastic spectral acceleration at the initial fundamental period of the structure as the primary intensity measure complemented by one or two newly proposed nonlinear SDOF-based ground motion intensity measures as secondary intensity measures.

1 INTRODUCTION

The development of a probabilistic seismic performance assessment framework for performance-based earthquake engineering is underway at the Pacific Earthquake Engineering Research (PEER) Center head-quartered at the University of California at Berkeley (http://peer.berkeley.edu). The PEER framework breaks down the formidable task of assessing probabilistically the decision variables related to a specific civil structure such as a building or a bridge into the following four sub-tasks according to the *Total Probability Theorem* of applied probability theory. Examples of decision variables to be evaluated probabilistically are: annual probability of exceeding a given limit-state, annual probability of earthquake damage exceeding a given dollar amount, or annual probability of repair time (or down time) exceeding a specified threshold.

(1) *Probabilistic Seismic Hazard Analysis* (PSHA) with the objective to compute for a given site the annual probability of exceeding any particular value of a specified ground motion intensity measure (IM).

(2) *Probabilistic Seismic Response Analysis Conditional on IM* with the objective to determine the probability distribution of any pertinent engineering demand parameters (EDP's) conditioned on IM.

(3) *Probabilistic Failure (or Damage) Analysis Conditional on the EDP*('s) also called *Fragility Analysis* with the objective to compute the probability of exceeding a specified physical damage-state (or limit-state) given the EDP('s).

(4) *Probabilistic Analysis of Repair Cost and/or Repair Time Conditional on Damage State* with the objective of determining the "annual" probability distribution of decision variables such as repair cost and repair time for a given damage or failure mechanism.

1.1 *Probabilistic seismic hazard analysis*

In probabilistic seismic hazard analysis, the mean/average annual rate/frequency (or Poisson rate), $\lambda_{IM}(z)$, of exceeding a particular threshold value, z, of a ground motion intensity measure, IM, is obtained as (Cornell 1968):

$$\lambda_{IM}(z) = \sum_{i=1}^{N_{flt}} \lambda_i \int_{R_i} \int_{M_i} P[IM > z | m, r] \cdot f_{M_i}(m) f_{R_i}(r) dm dr \quad (1)$$

where N_{flt} = number of causative faults; λ_i = mean annual rate/frequency of occurrence of earthquakes with magnitudes greater than a lower-bound threshold value, m_0, on fault i. Functions $f_{M_i}(m)$ and $f_{R_i}(r)$ denote the probability density functions (PDF) for magnitude (M_i) and site-to-source distance (R_i), respectively, given the occurrence of an earthquake on fault *i*. The conditional probability of IM exceeding the threshold value z given $M_i = m$ and $R_i = r$ corresponds to one minus the cumulative distribution function (CDF) of the IM attenuation (or predictive relationship of IM given seismological variables M and R) (Abrahamson and Silva, 1997; Campbell, 1997).

1.2 *Probabilistic seismic demand hazard analysis*

The mean annual frequency, $\lambda_{EDP}(d)$, of a given structural response parameter (or engineering demand parameter EDP) exceeding a specified threshold value d is obtained by convolving the probability distribution of the EDP in question conditioned on the seismological variables M and R and the ground motion intensity measure IM, $P[EDP > z|IM, M, R]$, with the seismic hazard, $\lambda_{IM}(z)$, as

$$\lambda_{EDP}(d) = \sum_{i=1}^{N_{flt}} \lambda_i \int_{R_i}\int_{M_i}\int_{IM} P[EDP > d|IM, M_i, R_i] \cdot f_{IM|M_i,R_i}(im|m,r) f_{M_i}(m) f_{R_i}(r) d(im)\, dm\, dr \tag{2}$$

where $f_{IM|M_i,R_i}(im|m,r)$ denotes the conditional probability density function of IM given M, R and the occurrence of an earthquake along seismic fault i. Assuming that the selected ground motion intensity measure IM renders EDP conditionally independent, given IM, of earthquake magnitude (M) and source-to-site distance (R)[1], i.e.,

$$P[EDP > d|IM, M_i, R_i] = P[EDP > d|IM], \tag{3}$$

Eq. (2) simplifies to

$$\lambda_{EDP}(d) = \int_{IM} P[EDP > d|IM] \cdot |d\lambda_{IM}(im)| \tag{4}$$

1.3 *Structural reliability analysis*

The probabilistic assessment of a specific structure requires the consideration of a number of potential damage states. Here, a damage state is defined as a particular stage of a specified failure mode (or failure mechanism). Typically, these damage states are characterized by mathematical damage-state or limit-state functions of the form

1. This condition of “conditional independence, given IM, of earthquake magnitude (M) and source-to-site distance (R)” is called the “sufficiency condition” by Cornell et al. (Luco and Cornell, 2003; Jalayer and Cornell, 2002).

$$Z_k = R_k - S_k \tag{5}$$

where R_k and S_k denote the resistance/capacity and load effect/demand, respectively, related to the *k*-th damage state and Z_k is the corresponding safety margin. The safety margin Z_k is a random variable due to (1) the uncertain capacity term R_k stemming from the inherent randomness of material, mechanical and geometric properties defining the structure, (2) the modeling uncertainty associated with the capacity term R_k and the limit-state function as a whole, (3) the intrinsic variabilities of the demand term S_k, and (4) the intrinsic variabilities of the demand as a whole beyond the demand term S_k used in formulating the limit-state function (i.e., missing demand variables affecting the physical limit-state under consideration). Traditionally, the above sources of uncertainty (1), (2), and (4) are modeled and quantified through the probability of exceeding the k-th damage- or limit-state conditioned on the demand variable S_k, namely

$$P[Z_k < 0|S_k = s] \tag{6}$$

The probabilistic analysis involved in evaluating the above conditional probability of limit-state exceedance is traditionally called *fragility analysis*. The only way to assess capacity model uncertainty is to compare model predictions with real-world observations, either in the field or in the laboratory, and to perform statistical model assessment for example using Bayesian statistical methods (Gardoni et al. 2002).

The mean annual frequency, λ_{LS_k}, of exceeding the *k*-th limit- or damage-state of a specific civil structure is obtained by convolving the probability of exceeding the *k*-th limit-state conditional on the demand variable (*EDP*), $P[Z_k < 0|EDP = d]$, with the demand hazard, $\lambda_{EDP}(d)$, as

$$\lambda_{LS_k} = \int_{EDP} P[Z_k < 0|EDP = d] \cdot |d\lambda_{EDP}(d)| \tag{7}$$

1.4 *Probabilistic assessment of decision variables*

The above probabilistic analysis can be extended to the decision variables (DV’s) (e.g., life safety, downtime, and dollar losses) related to a specific civil structure. In order to do so, we first need to focus on a specific prevalent damage/failure mechanism FM_i such as pier(s)/column(s) flexural and/or shear failure for a reinforced concrete bridge or a building, and characterize the stage of formation of this damage mechanism through a sequence of discrete damage states[2] defined by limit-state functions Z_k, $k = 1, \ldots, n$. For example, we could have five discrete levels of pier/column damage such as (Hose

2. In contrast with a continuum of damage states, for the sake of simplifying the problem through discretization.

and Seible, 1999): Level 1 = no visible damage characterized by onset of barely visible cracks, Level 2 = minor damage consisting of visible cracking due to yielding of reinforcement, Level 3 = moderate damage described by the onset of inelastic deformation and concrete spalling (initiation of local mechanism), Level 4 = major damage defined by large crack widths (greater than 2 mm) and extensive spalling (full development of local mechanism), and Level 5 = local failure/collapse characterized by large residual deformations such as buckling and rupture of reinforcement and crushing of the concrete core, accompanied by strength degradation. Thus, for the specified damage mechanism FM_i, the mean annual frequency, $\lambda_{DV}^{FM_i}(z)$, of decision variable DV exceeding a threshold level *z* is obtained by convolving (in discrete form) the probability of DV exceeding level *z* conditional on the *k-th* damage-state being reached or exceeded but without the *(k+1)-th* damage-state being reached, with the hazard of being between the *k-th* and *(k+1)-th* damage-state, $\lambda_{LS_k} - \lambda_{LS_{k+1}}$, as

$$\lambda_{DV}^{FM_i}(z) = \sum_{k=1}^{n-1} P[DV > z | (Z_k \leq 0) \cap (Z_{k+1} > 0)] \cdot \cdot (\lambda_{LS_k} - \lambda_{LS_{k+1}}) + P[DV > z | Z_n < 0] \cdot \lambda_{LS_n} \quad (8)$$

The above must be repeated for each of the N_{FM} prevalent damage/failure mechanisms FM_i, i = 1, ..., N_{FM}. The mean annual frequency, $\lambda_{DV}(z)$, of decision variable DV exceeding a threshold level z accounting for all N_{FM} prevalent damage/failure mechanisms (e.g., column failure, foundation failure, abutment failure, superstructure failure, ...) will then be obtained through combination of the individual mean annual frequencies, $\lambda_{DV}^{FM_i}(z),\ i = 1, \ldots, N_{FM}$, corresponding to the N_{FM} failure mechanisms accounting for their statistical dependency. In the terminology of structural reliability (Ditlevsen and Madsen, 1996), the problem of determining $\lambda_{DV}^{FM_i}(z)$ or $P[(DV)_{year}^{FM_i} > z]$ is a structural component reliability problem, while the problem of finding $\lambda_{DV}(z)$ or $P[(DV)_{year} > z]$ is a system reliability problem.

1.5 *Research objectives*

An issue of paramount importance in the PEER probabilistic framework is the choice of the IM that can be taken as either a scalar or a vector quantity. The choice of IM has a profound impact on the simplifying assumptions and methods that can be used to evaluate accurately and efficiently the PEER hazard integral, Eq. (8), which aggregates the results of the four sub-tasks defined above, for families of civil structures such as buildings and bridges.

In the present study, the primary ground motion intensity measure is taken as the 5 percent damped elastic spectral acceleration, $S_a(T_0, \xi_0 = 0.05)$, at the (initial) fundamental period T_0 of the structure as suggested by a significant body of previous work (Kennedy et al., 1984; Sewell 1998; Shome et al., 1998). This paper presents the results of a comprehensive parametric probabilistic/statistical study with the objective to identify, within a class of existing and newly defined ground motion intensity measures IM, the optimum ones from the viewpoint of their “efficiency” in reducing the dispersion, after conditioning the seismic input records on $S_a(T_0)$ and IM, of a variety of engineering demand parameters (EDP’s) indicative of structural damage. The newly defined ground motion intensity measures are based on various features of the nonlinear seismic response of generic nonlinear SDOF systems. In other words, the objective here is to identify optimum post-elastic ground motion intensity measures complementing $S_a(T_0, \xi_0 = 0.05)$ which is a measure of peak elastic structural response imposed by the ground motion on the structure.

2 GROUND MOTION DATABASE

The database used in this study consists of 1851 recordings from 157 earthquakes. These recordings are from world-wide shallow crustal earthquakes near active plate margins. Subduction and inter-plate events are excluded. Event dates range from the 1935 Helena, Montana earthquake to the 1999, Chi-Chi, Taiwan, and Kocaeli and Duzce, Turkey, earthquakes. Removed from the data set for this study were low-amplitude recordings (peak horizontal acceleration < 0.1g) and records with high-pass filter frequency > 0.2 Hz or low-pass filter frequency < 10 Hz. These removals reduced the data set to 881 recordings, only horizontal ground motion components (one or two per station), from 80 events. The distribution of magnitude (M) and closest site-source distance (R) parameters for the full data set is shown in Figure 1. Most of the time histories used in this study can be obtained from the PEER strong motion database at the PEER website (http://peer.berkeley.edu).

Seismological and site variables were compiled for each recording in the database. These variables include magnitude, site-source distance, focal mechanism (i.e., reverse-slip vs. strike-slip), local site condition (i.e., soil site vs. rock site), and near-fault directivity conditions (forward vs. neutral directivity).

Ground motion intensity measures were compiled for each of the 881 recordings in the database. These parameters include peak ordinates from time histories of horizontal shaking (PGA, PGV, PGD), Arias intensity (AI), significant duration as derived from Husid plot (5-95% normalized Arias intensity), mean period (T_{mean}) as defined by Rathje et al. (1998), 5 percent damped elastic spectral pseudo-acceleration for periods of T_0 = 0.2, 0.5, 1.0, 2.0, 3.0 sec, and average spectral acceleration between T_0 and $2T_0$,

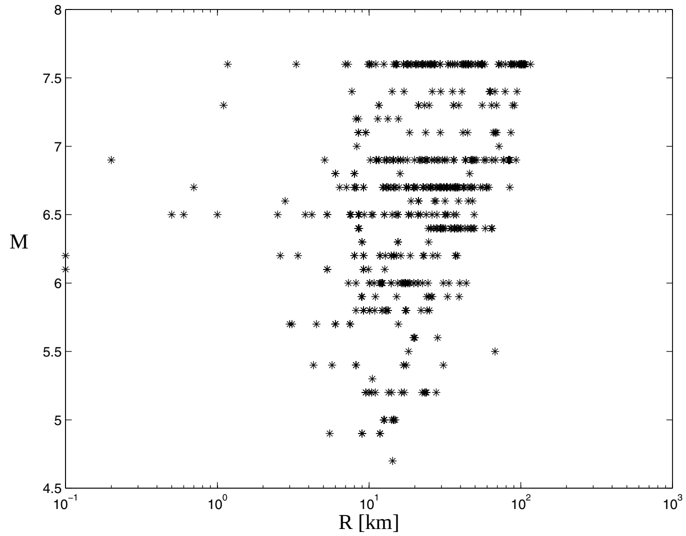


Figure 1. Magnitude - closest distance distribution of 881 “qualified” earthquake records.

$\bar{S}_a(T_0, 2T_0)$. Figure 2 shows the 16-percentile, median, mean, and 84-percentile elastic pseudo-acceleration response spectra for the ensemble of 881 “qualified” earthquake records. The median spectrum is used as target spectrum (or target primary ground motion intensity measure). Thus, for each ground motion time history, five different scaling factors are determined to match the median spectral ordinates for that time history to the five target S_a values at T_0 = 0.2, 0.5, 1.0, 2.0, 3.0 sec. In order to avoid excessive scaling, which can lead to unrealistic time histories, we discard time histories that require scaling factors less than 0.30 or greater than 3.30 at any one of the five spectral periods. Imposition of this condition reduces the 881 recordings in the data set (only horizontal components) to 550 individual earthquake time histories.

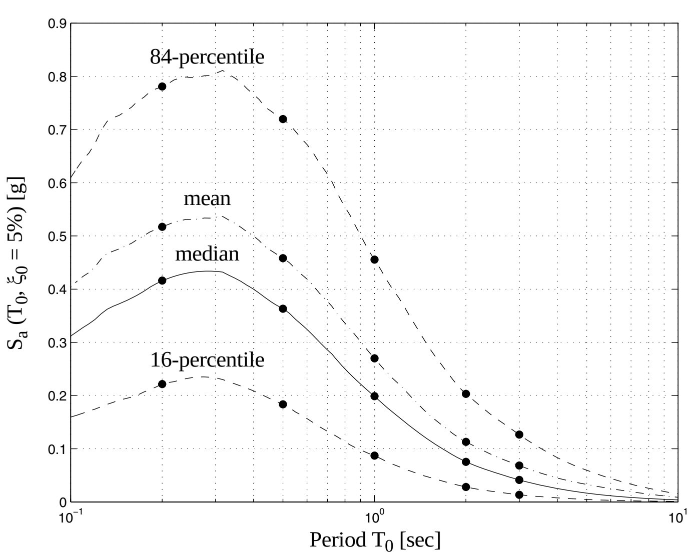


Figure 2. Median, mean, 16-percentile, and 84-percentile pseudo-acceleration response spectra for ensemble of 881 “qualified” earthquake records.

3 STRUCTURAL MODELS, PARAMETERS AND RESPONSE PARAMETERS

Only nonlinear inelastic SDOF systems are used as structural models in this study. Three hysteretic models are considered, namely the bilinear inelastic, Clough’s stiffness degrading, and slip models (Fig. 3). These inelastic SDOF dynamic systems are characterized by the following system parameters: initial period $T_0 = 2\pi\sqrt{m/k_0}$ [sec], damping ratio $\xi_0 = c/(2\sqrt{k_0 m})$, normalized strength $C_y = R_y/(mg)$, and strain hardening ratio $\alpha = k_p/k_0$, where m = mass, k_0 = initial or pre-yield stiffness, c = damping coefficient, R_y = yield strength, g = acceleration of gravity, and k_p = post-yield stiffness.

The equations of motion, energy balance equations and power balanced equations of the nonlinear SDOF systems subjected to earthquake ground motions are integrated using a piecewise linear exact integration algorithm. Three types of response parameters (or engineering demand parameters EDP) are considered in this study. Response parameters of the first type are based on the force-deformation response of the system and include maximum displacement ductility ($\mu = \mathrm{Max}(|u(t)|/U_y)$), maximum normalized plastic deformation range ($\Delta^*_{p,\,max} = \underset{i}{\mathrm{Max}}(|\Delta u_{p,\,i}/U_y|)$, see Fig. 3, residual displacement ductility ($\mu_{res} = |u_{res}|/U_y$), cumulative displacement ductility ($\mu_{acc} = (\sum_{i=1}^{N}|\Delta u_{p,\,i}/U_y|)$ +1), number of positive/negative yield excursions, number of yield reversals ($N_{y,rev}$). Response parameters of the second type derive from the energy balance equation and include the normalized maximum input energy over the duration t_d of the ground motion record,

$$E^*_{I,\,max} = \{\underset{0 \le t \le t_d}{\mathrm{Max}} E_I(t)\}/(R_y U_y), \tag{9}$$

and the normalized hysteretic energy dissipated over the duration of the ground motion record,

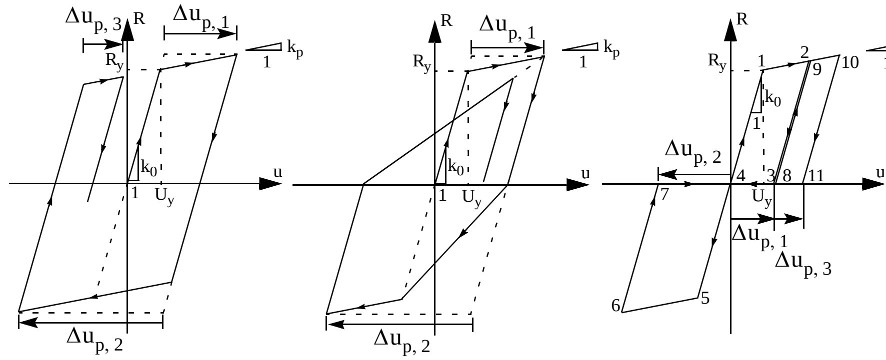
$$E^*_H = E_H(t_d)/(R_y U_y), \tag{10}$$

where $U_y = R_y/k_0$ denotes the yield displacement. Response parameters of the third type are based on the power balance equation and include the maximum rate of normalized earthquake input energy,

$$P^*_{I,\,max} = \underset{0 \le t \le t_d}{\mathrm{Max}} \left[\frac{1}{(R_y U_y)}\dot{E}_I(t)\right] \quad [\mathrm{sec}^{-1}], \tag{11}$$

and the maximum rate of normalized hysteretic energy dissipation,

$$P^*_{H,\,max} = \underset{0 \le t \le t_d}{\mathrm{Max}} \left[\frac{1}{(R_y U_y)}\dot{E}_H(t)\right] \quad [\mathrm{sec}^{-1}], \tag{12}$$



(a) Bilinear Model (b) Clough's Stiffness Degrading Model (c) Slip Model

Figure 3. SDOF hysteretic models

where a dot over a symbol denotes one differentiation with respect to time. The last two response parameters are indicative of how fast the earthquake input energy is imparted to and dissipated by the structure through both viscous damping and inelastic action.

The nonlinear SDOF response parameters defined above can be viewed as damage indices. They have been computed for the three nonlinear SDOF systems shown in Fig. 3 at the five initial periods T_0 = 0.2, 0.5, 1.0, 2.0, 3.0 sec, for several values of the normalized strength C_y, and for each of the 550 earthquake records scaled to the median elastic spectrum shown in Fig. 2. The results presented here all correspond to a damping ratio of $\xi_0 = 0.05$ and a strain hardening ratio of $\alpha = 0$ (i.e., elastic-perfectly plastic).

Two types of response analysis were performed, namely (1) forward/direct analysis, and (2) inverse/iterative analysis. In a forward/direct analysis, the nonlinear response parameters of a given nonlinear SDOF system with specified structural parameters (T_0, ξ_0, R_y, and α) are computed for a given earthquake ground motion record. In an inverse/iterative analysis, the minimum strength level required ($C_y = R_y/(mg)$) to limit a specified inelastic response parameter to a given value is determined through an iterative process. For example, constant-ductility spectra, which represent very useful design tools, are the product of an inverse analysis, consisting of determining, for a given SDOF system, the minimum strength level required, as measured by the yield acceleration A_y (= R_y/m), to limit the maximum displacement ductility μ of this system to a specified value (e.g., μ = 2, 4, 6, 8). In this study, the targeted response parameters considered are the maximum displacement ductility (μ), the number of yield reversals ($N_{y,rev}$), the normalized hysteretic energy dissipated (E_H^*), and the maximum rate of normalized hysteretic energy dissipation, ($P_{H,\max}^*$).

4 NONLINEAR SDOF-BASED GROUND MOTION INTENSITY MEASURES

In this study, particular attention is placed on nonlinear SDOF-based ground motion intensity measures which have the potential to complement effectively the spectral acceleration, $S_a(T_0, \xi_0 = 0.05)$, at the fundamental period of a structure (used as primary ground motion intensity measure) in evaluating the hazard integral, Eq. (8), for civil structures modeled as nonlinear MDOF systems. A generic nonlinear SDOF-based ground motion intensity measure is defined as

$$F_{R=r} = \frac{C_y^{R=r}}{C_y^{\text{elastic}}} \tag{13}$$

where R denotes a nonlinear SDOF response parameter (e.g., ductility μ, normalized hysteretic energy dissipated E_H^*, ...) of a generic nonlinear inelastic SDOF system and r represents a specific/target value of this inelastic response parameter (e.g., $\mu = 6$) taken by the generic system when subjected to the ground motion record considered. In this study, the generic system is taken as the bilinear inelastic SDOF system with zero strain hardening ratio, i.e., elastic-perfectly plastic SDOF system. Thus, $F_{R=r}$ is defined as the ratio of the minimum yield strength required, $C_y^{R=r}$, to limit the inelastic response parameter R to r (when subjected to the ground motion record considered) to the minimum yield strength required for the system to remain elastic, C_y^{elastic}. Notice that $F_{R=r}$ is smaller than or equal to one, i.e.,

$$F_{R=r} \le 1 \tag{14}$$

The closer $F_{R=r}$ is to one, the more effective the ground motion is in producing the target level r of the nonlinear response parameter R in a structure. Of particular interest in this study will be the cases where the inelastic response parameter R is defined as the maximum displacement ductility μ, the normalized hysteretic energy dissipated E_H^*, the number of yield reversals $N_{y,rev}$, and the maximum rate of normalized hysteretic energy dissipation $P_{H,\max}^*$. Thus, F_μ, $F_{E_H^*}$, $F_{N_{y,rev}}$, and $F_{P_{H,\max}^*}$, are measures of the effectiveness of a ground motion record to produce a given level of displacement ductility, hysteretic energy dissipation, yield reversals, and maximum rate of hysteretic energy dissipation, respectively, in a structure. From the normalized equation of motion of a nonlinear SDOF system, it can be shown that these new IM's are independent of the scaling of the ground motion and, therefore, are relative intensity measures. These new IM's allow to quantify the effects of a ground motion record on an inelastic system above and beyond its effects on peak elastic response as measured by the elastic spectral displacement or spectral acceleration at the initial period of the system.

As an illustration of the results of the statistical inverse analysis, Fig. 4 shows the histograms and coefficient-of-variation (c.o.v.) of the yield strength coefficient required, C_y^{μ}, for a maximum displacement ductility response of $\mu = 2$ and 8 in the case of a bilinear SDOF system with parameters given in the figure. The median value of C_y^{μ}, $\tilde{C}_y^{\mu}$, is also given in the figure for both cases. An example of results

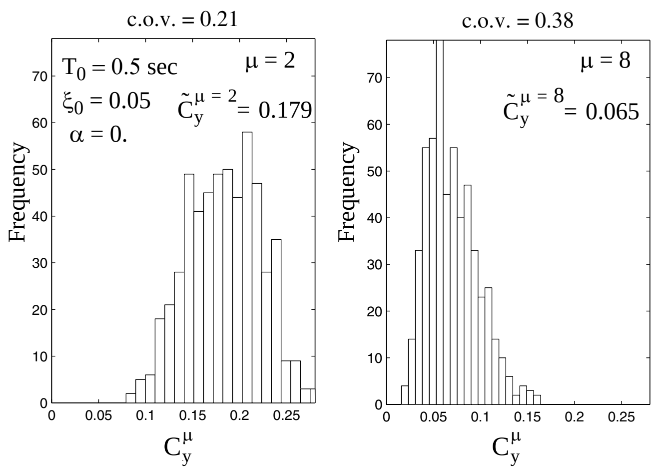


Figure 4. Histograms of C_y required for $\mu = 2$ and 8

obtained from the forward/direct analysis is given in Fig. 5 which displays the histograms and coefficient of variation (c.o.v.) of the maximum displacement ductility response μ for the same inelastic SDOF system as in Fig. 4 (with strength levels $\tilde{C}_y^{\mu=2}$ and $\tilde{C}_y^{\mu=8}$).

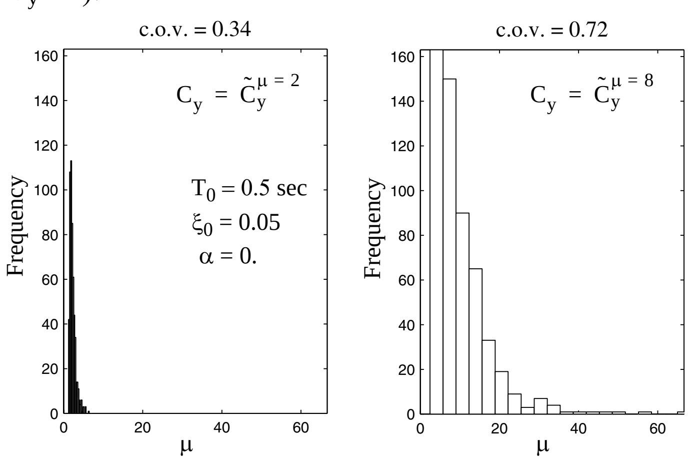


Figure 5. Histograms of maximum displacement ductility μ

5 OPTIMUM POST-ELASTIC GROUND MOTION INTENSITY MEASURES

A parametric statistical study is performed to systematically analyze the statistics of nonlinear SDOF response parameters and the statistical correlation between nonlinear SDOF response parameters and both seismological variables and ground motion intensity measures. In contrast to other past and current studies of this type (Riddell 1994, Shome et al. 1998, Miranda 1993, 2000), there is no *a priori* binning of the strong motion database, and multiple inelastic SDOF response parameters are considered. Again, in this study, the 5 percent damped elastic spectral acceleration at the initial period of the SDOF system, $S_a(T_0, \xi_0 = 0.05)$, is taken as the primary ground motion intensity measure. Extensive statistical correlation analysis was performed between: (1) nonlinear SDOF response parameters and seismological variables, (2) nonlinear SDOF response parameters and IM's (both traditional and new), (3) different nonlinear SDOF response parameters of the same hysteretic model, and (4) a given nonlinear SDOF response parameter for different hysteretic models. The degree of correlation is measured by the sample correlation coefficient ρ. One of the primary objectives of these correlation studies was to determine which traditional ground motion parameters correlate best with inelastic SDOF response parameters indicative of damage or, in other words, to identify the most damaging features of earthquake ground motions. Another primary objective was to investigate whether the newly defined nonlinear SDOF-based ground motion intensity measures correlate better with various nonlinear SDOF response parameters over a wide range of system parameters than traditional engineering ground motion parameters. As an illustration of the correlation analysis, Fig. 6 displays scatter diagrams (or correlograms) and correlation coefficients (ρ) of nonlinear SDOF response parameters and seismological variables and traditional IM's. These scatter diagrams are based on the set of 550 earthquake records scaled to the median spectrum shown in Fig. 2. Illustrative scatter diagrams and correlation coefficients of nonlinear SDOF response parameters and two of the newly defined ground motions intensity measures $F_{\mu=8}$ and $F_{E_H^*=100}$ are shown in Fig. 7.

An illustrative sample of the complete set of correlation coefficients obtained in this study is represented graphically in Fig. 8 in the form of bar diagrams. The correlation between the new IM's $F_{\mu=x}$, $F_{E_H^*=x}$, $F_{N_{y,rev}^*=x}$, $F_{P_{H,max}^*=x}$ and nonlinear response parameters depends moderately on the target value x as shown in Fig. 9. Some of the significant findings of this correlation analysis are:

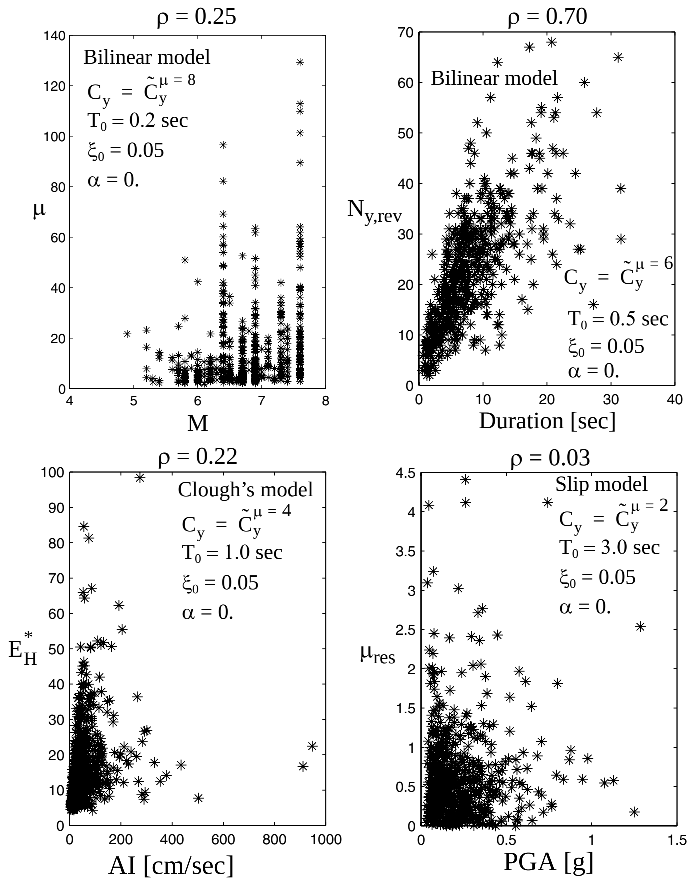


Figure 6. Correlograms of nonlinear SDOF response parameters and seismological variables and traditional ground motion intensity measures

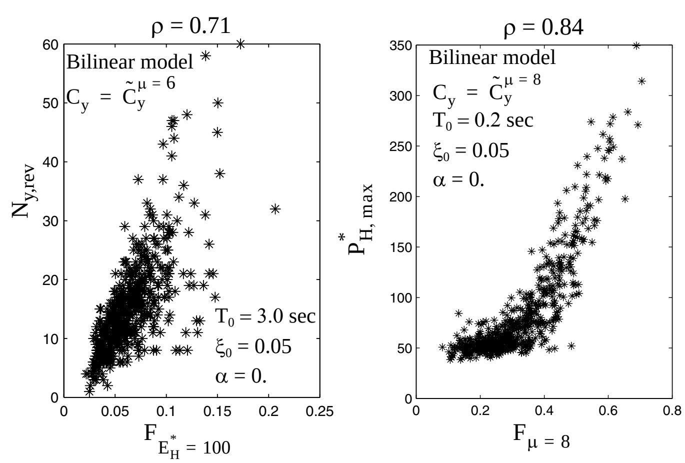


Figure 7. Correlograms of nonlinear SDOF response parameters and newly defined ground motion intensity measures

- As the initial period T_0 increases, there are progressively less pairs of significantly correlated traditional IM's and inelastic SDOF response parameters. None of the traditional IM's correlates well with all or even a majority of inelastic SDOF response parameters consistently across all T_0 values and strength levels considered. The correlation between traditional IM's and inelastic SDOF response parameters is very much period dependent.

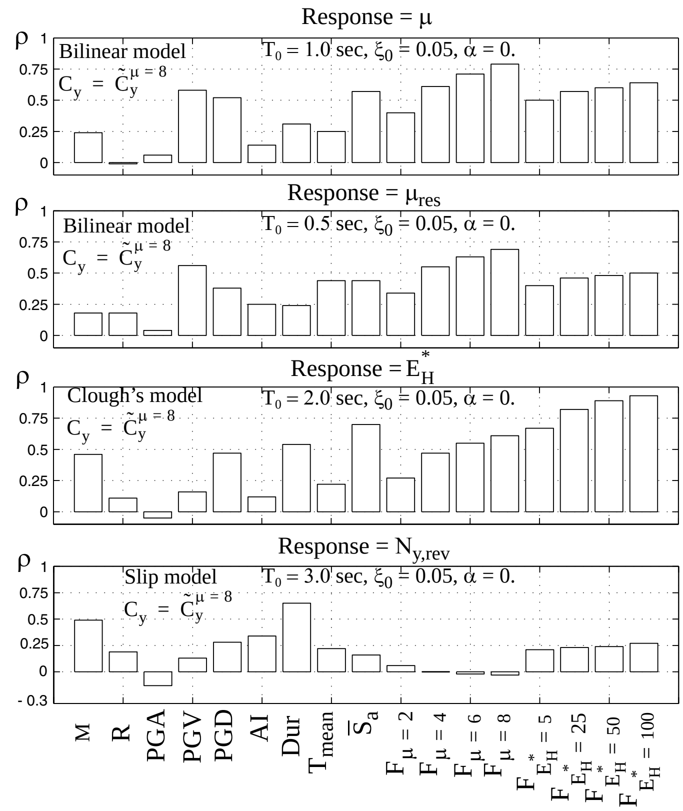


Figure 8. Comparison of correlation of seismological variables and ground motion IM's (traditional and new) with inelastic SDOF response parameters

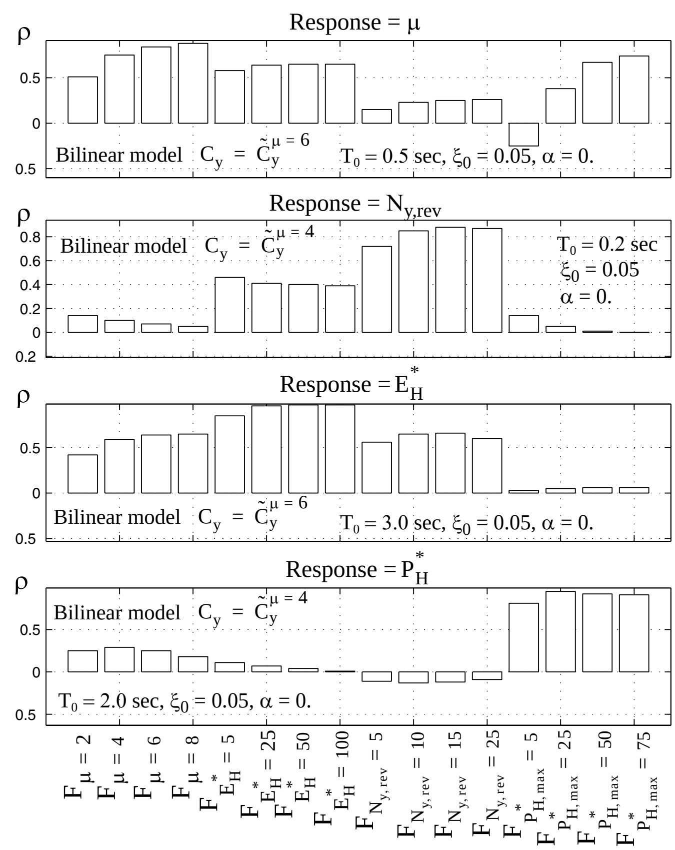


Figure 9. Comparison of correlation of new ground motion IM's with inelastic SDOF response parameters

- The best traditional IM's in terms of correlation with nonlinear response parameters are (1) the average elastic spectral acceleration $\bar{S}_a(T_0, 2T_0)$, which correlates fairly well with energy-based response parameters and fairly well with deformation based response parameters for all T_0 values considered, and (2) PGV, which correlates well with power based response parameters across all T_0 values considered.
- In general, the newly defined IM's are better correlated to the nonlinear SDOF response parameters than the traditional IM's. They achieve a more consistent correlation across all periods and strength levels for the three hysteretic structural models considered. In particular, the new IM's $F_{\mu = 8}$ and $F_{E_H^* = 100}$ exhibit high correlation with a majority of inelastic SDOF response parameters across all T_0 values considered.

Probabilistic conditioning with respect to the new IM's above and beyond the conditioning with respect to the spectral acceleration at T_0, $S_a(T_0, \xi_0 = 0.05)$, allows to further reduce the dispersion (scatter) of nonlinear response parameters as illustrated in Fig. 10. Considering the desired attributes of optimum ground motion intensity measures (i.e., their complementarity and therefore low correlation, their efficiency in reducing the dispersion of a set of structural response parameters) and based on the results of the present study, the following two- and three-component vectors of ground motion intensity measures (i.e. vector-valued IM's) are proposed: (1) $\{S_a(T_0), F_{E_H^* = 100}\}$, (2) $\{S_a(T_0), F_{\mu = 8}, F_{N_{y,rev} = 15}\}$, and (3) $\{S_a(T_0), F_{E_H^* = 100}, F_{P_{H,max}^* = 75}\}$.

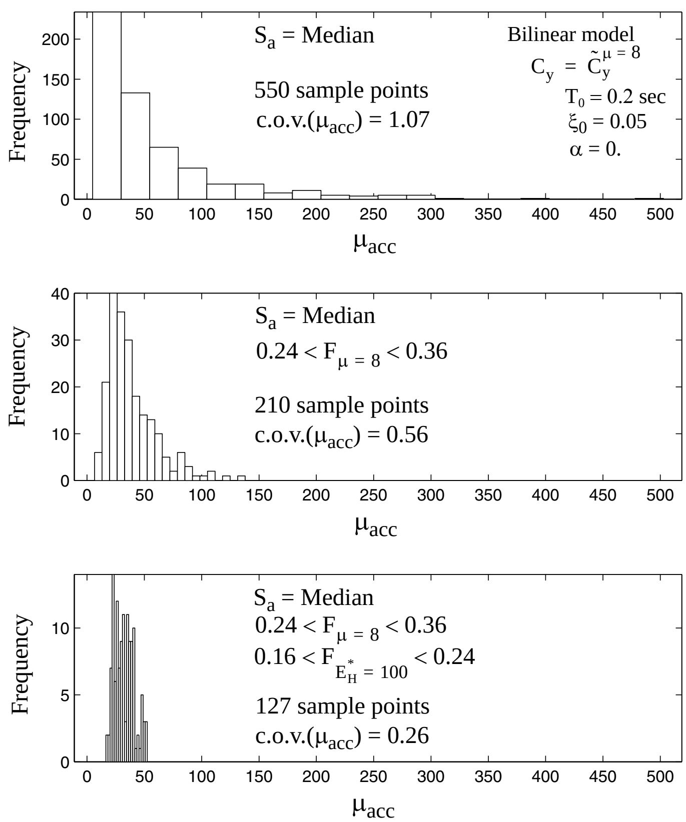


Figure 10. Reduction in dispersion of response parameter μ_{acc} when conditioned on $S_a(T_0)$, $F_{\mu = 8}$, and $F_{E_H^* = 100}$ (bilinear model with T_0 = 0.2 sec and $C_y = \tilde{C}_y^{\mu = 8}$)

The proposed intensity measures have been validated at the nonlinear SDOF level, across several hysteretic models, and remain to be validated at the nonlinear MDOF level.

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