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Performance Prediction Equations for Linear Planar Structural Systems: Concept, Formulation, and Validation

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This paper presents and validates an analytical formulation, denoted as Performance Prediction Equations (PPEs), that relates the seismic response engineering demand parameter (EDP) of buildings to earthquake parameters such as magnitude, epicentral distance, and type of faulting. PPEs are conceptually novel and can be readily included in any hazard calculation program to directly estimate EDP hazard curves. The PPEs presented herein are based on the linearization of response spectrum analysis (RSA) formulation for estimation of the seismic response of multi-degree-of-freedom (MDOF) models for planar structural systems. Equations for mean and variance are provided for floor displacement, interstory drift ratio, and normalized base shear. The input parameters needed to apply the proposed PPEs are the modal properties of the structural system and the selection of an existing ground motion model (GMM). The proposed PPEs are validated against simulated results using a set of planar building models and the Campbell-Bozorgnia 2014 GMM. The comparison confirms that the proposed PPEs provide an accurate estimate of the statistics of the said EDPs. [DOI: 10.1193/110716EQS194M]

INTRODUCTION

Probabilistic seismic demand analysis (PSDA) is part of the performance-based seismic assessment methodology (Cornell and Krawinkler 2000); it allows the estimation of the mean annual frequency of exceedance of a specified threshold of the engineering demand parameter (EDP) of interest (denoted as λ_{EDP}). PSDA is traditionally exercised in two steps; in the first step, probabilistic seismic hazard analysis (PSHA) (Cornell 1968, McGuire 1995) is used to calculate the annual probability of exceedance of a ground motion intensity measure (IM). The result of the first step is the IM hazard curve (denoted as λ_{IM}), which provides the relationship between the IM and λ_{IM} as shown in Equation 1, where n_s is the number of potential earthquake sources at the location of interest, ν_p is the average seismic rate of source p , θ represents the vector of all ground motion parameters (magnitude, distance, etc.), and $f_p(\theta)$ is the joint probability density function of the ground motion parameters for earthquake source p . In the second step, the IM hazard curve is integrated with a function that describes the IM-EDP relationship, resulting in the EDP hazard curve, λ_{EDP} , see Equation 2. The EDP hazard curve allows the evaluation of the performance of the structure—in EDP format—at the return period of interest. The IM-EDP distribution in Equation 2 is

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obtained through multiple nonlinear time history analyses of the structure (see [Baker and Cornell 2005](#) for a detailed description of the available methods), and it relies on the selection of an intensity measure that appropriately represents the response of the structure.

$$\lambda_{IM}(im) = \sum_{p=1}^{n_s} \nu_p \int_{all \theta} P(IM > im | \theta) f_p(\theta) d\theta \quad (1)$$

$$\lambda_{EDP}(edp) = \int_{im} P(EDP > edp | IM = im) | d\lambda_{IM}(im) | \quad (2)$$

PSDA is exercised for a given building and location; the information needed to exercise PSDA is the seismicity of the area, the distribution of IM given ground motion parameters—which is given by any ground motion model (GMM)—and the IM-EDP relationship. As this two-step approach highlights, the results are generally dependent on the selected IM. IM definition has shifted from characteristics of the ground motion alone (e.g., PGA or peak ground velocity [PGV]) to the currently used spectral acceleration, $S_a(T_1)$, which carries information about the ground motion and the fundamental period of the structure. Advanced IMs have been introduced over the years—see [De Biasio et al. 2014](#), [Luco and Cornell 2007](#), and [Tothong and Luco 2007](#), just to name a few—generally to improve the representation of the structural response, but the simplicity of calculation of $S_a(T_1)$ and its predominance in the engineering practice have allowed it to remain the IM of choice to this date. $S_a(T_1)$ is, after all, the selected intensity measure used to develop the ground motion models (GMMs) for the recent NGA-West2 Project ([Bozorgnia et al. 2014](#)).

The strength of a ground motion intensity measure lies in its ability to link the structural response with the ground motion that caused it. Several methods have been developed to quantify this property ([Jalayer et al. 2012](#), [Kwong et al. 2015](#), [Luco and Cornell 2007](#)). Luco and Cornell define a *good* intensity measure as one that is both “efficient” and “sufficient” in estimating the response variable. Efficiency is represented by low dispersion in the estimated EDP using that IM, which implies that fewer nonlinear analyses are needed to estimate means and distributions of the EDP; sufficiency indicates that the intensity measure alone completely characterizes the response, that is, distinct sets of ground motions will return the same IM-EDP distribution.

The use of the two-step approach to develop an EDP hazard curve λ_{EDP} has several advantages, including the simplicity of its implementation, the modularity of its computation in which hazard analysis is separated from seismic structural analysis, and the existence of several established GMMs that estimate the probabilistic distribution parameters for S_a . The use of IM as an intermediary quantity, however, inevitably introduces a source of bias in this calculation. This paper is part of a project that aims at the development of Performance Prediction Equations (PPEs), that is, EDP models that describe the distribution of the structural response as a function of the causing earthquake parameters. A first step in this direction can be attributed to [Bozorgnia et al. 2010a](#) and [2010b](#), who statistically developed a GMM (or prediction equation, using the previous terminology) for an inelastic single-degree-of-freedom (SDOF) system,

which can represent the behavior of a simplified realistic structure. In addition, [Hancock et al. 2008](#) used a selected set of 1,656 recorded accelerograms to fit a generalized functional form to the distribution of several damage measures for a specific code-conforming eight-story building. This paper summarizes the authors' efforts in developing applied formulae (i.e., PPEs) that calculate the response statistics, that is, mean and standard deviation, for any earthquake scenario, based on the complete quadratic combination (CQC; [Wilson et al. 1981](#)), on the assumption of elastic behavior of the structure, on the knowledge of the structural mode shapes, and on the selection of an existing GMM ([Campbell and Bozorgnia 2014](#) is used in this study). The analytical formulation presented herein simplifies the PSDA two-step approach and provides a direct mean to calculate λ_{EDP} without requiring additional regression analyses. The assumptions and analytical formulation of the approach are presented alongside validation tests using multi-degree-of-freedom (MDOF) systems. This concept paper does not intend to provide a comprehensive framework, but to introduce the reader to a new approach. Because of the assumptions necessary to develop the analytical formulation, these results should be used within their limitations of elastic or quasi-elastic behavior and for response parameters that are accurately estimated using the CQC method. Within this setting, based on the authors' experience and investigation, the CQC method can accurately estimate the response of MDOF systems used in this study compared with analytical results obtained from modal response history analysis in a wide range of ground motion regimes. The proposed equations, together with the practical assumption of lognormal distribution of the response (e.g., [Cornell et al. 2002](#) and [Hancock et al. 2008](#)), can be readily implemented in seismic hazard curve development programs to provide the direct calculation of the EDP hazard curves according to Equation 3, and these results will soon be included in future publications.

$$\lambda_{EDP}(edp) = \sum_{i=1}^n \nu_i \int_{all \theta} P(EDP > edp | \theta) f_i(\theta) d\theta \quad (3)$$

METHODOLOGY

The goal of this study is to present the concept and associated equations (i.e., PPE) to obtain the distribution of the structural response (i.e., EDP) to seismic excitation as a function of earthquake parameters (e.g., moment magnitude, epicentral distance, type of faulting, etc.). The many ways for developing such equations include regressing a complicated functional form to the response of building structures to a catalog of earthquakes similar to how GMMs are developed. The simplest way to develop PPEs, however, is by developing analytical formulae that describe EDP as a function of an IM, considering that the statistics of the IM with respect to the earthquake parameters can be calculated using a GMM. A well-known method that connects IM and EDP is response spectrum analysis (RSA), which combines the maximum seismic response of each vibration mode into a single value. When the CQC ([Wilson et al. 1981](#)) is used to combine the modal displacements, the formula for the total displacement at each story (denoted as δ_k) is as shown in Equation 4. In this equation, k is the story number, i is the mode number, $\Gamma_i = \int_0^1 \Phi_i(x) dx / \int_0^1 \Phi_i^2(x) dx$ is the modal participation factor for mode i (from [Chopra 2012](#), where x is the parameter representing the height of the structure in normalized fashion), $S_{a,i} = S_a(T_i)$ is the spectral acceleration at the i^{th} modal period,

$\Phi_{i,k}$ is the i^{th} mode shape of the structure at story k , $\omega_i = 2\pi/T_i$ is the circular frequency of mode i , and ξ is the damping ratio ($\xi = 0.05$).

$$\delta_k = \sqrt{\sum_{i=1}^n \sum_{j=1}^n \delta_{i,k} \alpha_{T_i, T_j} \delta_{j,k}} = \sqrt{\sum_{i=1}^n \sum_{j=1}^n \Gamma_i \Phi_{i,k} \frac{S_{a,i}}{\omega_i^2} \alpha_{T_i, T_j} \Gamma_j \Phi_{j,k} \frac{S_{a,j}}{\omega_j^2}}$$

$$\alpha_{T_i, T_j} = \frac{8 \xi^2 \left(1 + \frac{T_i}{T_j}\right) \left(\frac{T_i}{T_j}\right)^{\frac{3}{2}}}{\left[1 - \left(\frac{T_i}{T_j}\right)^2\right]^2 + 4 \xi^2 \frac{T_i}{T_j} \left(1 + \frac{T_i}{T_j}\right)^2} \quad (4)$$

An alternative way to combine the modal responses is the square root of the sum of the squares (SRSS) combination, which does not consider the correlation between periods but results in a simpler formulation. The CQC combination was selected to provide the most general formulation, also applicable to cases in which the vibrational periods are close. The modal combination presented in Equation 4 establishes a nonlinear relationship between the response variable δ_k and the vector of spectral accelerations $\mathbf{S}_a = [S_{a,1}, S_{a,2}, \dots, S_{a,n}]$. Using the first-order second-moment (FOSM) approximation reported in Appendix, it is possible to obtain a formulation for the mean and variance of the response parameter δ_k using the statistics of the independent variable $\mathbf{S}_a$. Assuming that the floor displacement is lognormally distributed, Equation 4 can be reformulated, as shown in Equation 5. This formulation, which describes $\ln \delta_k$ as a function of $\mathbf{S}_a$, provides an intuitive—and approximate—estimate of median of $\ln \delta_k$ results that consistently overestimate the true CQC response by almost 30%, as shown in De Bortoli et al. 2017. For this reason, it is preferable to reformulate Equation 4 to highlight the relationship between $\ln \delta_k$ and $\ln \mathbf{S}_a$, rather than $\mathbf{S}_a$, as seen in Equation 6.

$$\ln \delta_k = \frac{1}{2} \ln \sum_{i=1}^n \sum_{j=1}^n \frac{\Gamma_i \Phi_{i,k}}{\omega_i^2} S_{a,i} \alpha_{T_i, T_j} \frac{\Gamma_j \Phi_{j,k}}{\omega_j^2} S_{a,j} = f_k(S_{a,1}, S_{a,2}, S_{a,3}, \dots, S_{a,n}) \quad (5)$$

$$\ln \delta_k = \frac{1}{2} \ln \sum_{i=1}^n \sum_{j=1}^n \frac{\Gamma_i \Phi_{i,k}}{\omega_i^2} e^{\ln S_{a,i}} \alpha_{T_i, T_j} \frac{\Gamma_j \Phi_{j,k}}{\omega_j^2} e^{\ln S_{a,j}}$$

$$= g_k(\ln S_{a,1}, \ln S_{a,2}, \ln S_{a,3}, \dots, \ln S_{a,n}) \quad (6)$$

The FOSM approximation—see Equation 23 in the Appendix—can, therefore, be applied to the function g_k to obtain the statistics of $\ln \delta_k$. We assume that the inclusion of three modes (i.e., $n = 3$) sufficiently captures all important dynamic effects for EDPs considered in this study. Assuming that δ_k is lognormally distributed, its natural logarithm is normally distributed with mean $\mu_{\ln \delta_k}$ and variance $\sigma_{\ln \delta_k}^2$. When calculated at the mean value of the natural logarithm of $\mathbf{S}_a$, denoted as $\boldsymbol{\mu}_{\ln \mathbf{S}_a} = [\mu_{\ln S_{a,1}}, \mu_{\ln S_{a,2}}, \mu_{\ln S_{a,3}}]$, the partial derivatives of g_k with respect to $\ln S_{a,i}$ are shown in Equation 7, where $A_{i,k} = \Gamma_i \Phi_{i,k} / \omega_i^2$. Subsequently, the mean and variance of $\ln \delta_k$ can be obtained as shown in Equation 8, in which $AA_{i,k} = A_{i,k} e^{\mu_{\ln S_{a,i}}} \sum_{t=1}^3 A_{t,k} \alpha_{T_i, T_t} e^{\mu_{\ln S_{a,t}}}$ and $CQCA_k = \sum_{i=1}^3 \sum_{j=1}^3 A_{i,k} A_{j,k} \alpha_{T_i, T_j} e^{\mu_{\ln S_{a,i}}} e^{\mu_{\ln S_{a,j}}}$.

Given Equation 8, the values needed to estimate the $\ln \delta_k$ statistics are the mean and covariance matrix of the normally distributed random vector $\ln S_a$. The logarithmic mean $\mu_{\ln S_{a,i}}$ and standard deviation $\sigma_{\ln S_{a,i}}$ are readily available using any GMM, and several empirical models have been developed to estimate the logarithmic correlation coefficient $\rho_{\ln S_{a,ij}}$ between spectral acceleration values. The analytical formulation proposed by Baker and Jayaram 2008 is used in this study.

$$\left. \frac{\partial g_k(\ln S_a)}{\partial \ln S_{a,i}} \right|_{\ln S_a = \mu_{\ln S_a}} = \frac{A_{i,k} e^{\mu_{\ln S_{a,i}}} \sum_{t=1}^3 A_{t,k} \alpha_{T_i, T_t} e^{\mu_{\ln S_{a,t}}}}{\sum_{i=1}^3 \sum_{j=1}^3 A_{i,k} A_{j,k} \alpha_{T_i, T_j} e^{\mu_{\ln S_{a,i}}} e^{\mu_{\ln S_{a,j}}}} = \frac{AA_{i,k}}{CQCA_k} \quad (7)$$

$$\begin{aligned} \mu_{\ln \delta, k} &= g_k(\mu_{\ln S_a}) = \frac{1}{2} \ln \left(\sum_{i=1}^3 \sum_{j=1}^3 A_{i,k} A_{j,k} \alpha_{T_i, T_j} e^{\mu_{\ln S_{a,i}}} e^{\mu_{\ln S_{a,j}}} \right) = \frac{1}{2} \ln (CQCA_k) \\ \sigma_{\ln \delta, k}^2 &= \sum_{i=1}^3 \sum_{j=1}^3 \left[\frac{\partial g_k(\ln S_a)}{\partial \ln S_{a,i}} \frac{\partial g_k(\ln S_a)}{\partial \ln S_{a,j}} \right] \bigg|_{\ln S_a = \mu_{\ln S_a}} \rho_{\ln S_{a,ij}} \sigma_{\ln S_{a,i}} \sigma_{\ln S_{a,j}} \\ &= \frac{1}{CQCA_k^2} \sum_{i=1}^3 \sum_{j=1}^3 AA_{i,k} AA_{j,k} \rho_{\ln S_{a,ij}} \sigma_{\ln S_{a,i}} \sigma_{\ln S_{a,j}} \end{aligned} \quad (8)$$

Similarly, it is possible to use RSA to determine the nonlinear relationship between interstory drift ratio **IDR** and the intensity measure S_a and use Taylor expansion to obtain $\mu_{\ln IDR, k}$ and $\sigma_{\ln IDR, k}$ as shown in Equations 9 and 10, in which $\Delta \Phi_{i,k} = (\Phi_{i,k} - \Phi_{i,k-1})/h_k$, h_k is the story height, $B_{i,k} = \Gamma_i \Delta \Phi_{i,k} / \omega_i^2$, $BB_{i,k} = B_{i,k} e^{\mu_{\ln S_{a,i}}} \sum_{t=1}^3 B_{t,k} \alpha_{T_i, T_t} e^{\mu_{\ln S_{a,t}}}$, $CQCB_k = \sum_{i=1}^3 \sum_{j=1}^3 B_{i,k} B_{j,k} \alpha_{T_i, T_j} e^{\mu_{\ln S_{a,i}}} e^{\mu_{\ln S_{a,j}}}$.

$$\begin{aligned} \ln IDR_k &= \frac{1}{2} \ln \sum_{i=1}^3 \sum_{j=1}^3 \frac{\Gamma_i \Delta \Phi_{i,k}}{\omega_i^2} e^{\ln S_{a,i}} \alpha_{T_i, T_j} \frac{\Gamma_j \Delta \Phi_{j,k}}{\omega_j^2} e^{\ln S_{a,j}} \\ &= g_k(\ln S_{a,1}, \ln S_{a,2}, \ln S_{a,3}) \end{aligned} \quad (9)$$

$$\begin{cases} \mu_{\ln IDR, k} = \frac{1}{2} \ln (CQCB_k) \\ \sigma_{\ln IDR, k}^2 = \frac{1}{CQCB_k^2} \sum_{i=1}^3 \sum_{j=1}^3 BB_{i,k} BB_{j,k} \rho_{\ln S_{a,ij}} \sigma_{\ln S_{a,i}} \sigma_{\ln S_{a,j}} \end{cases} \quad (10)$$

Finally, the approach presented herein can be used to obtain linearized equations for the base shear, V_b . The contribution of each mode to the total base shear, V_{bi} , where $i = 1, \dots, n$, can be calculated as the static participation factor V_{bi}^{st} multiplied by the pseudospectral acceleration $S_{a,i}$, as shown in Equation 11, where m is the story mass and assumed to be constant for all stories. Using RSA and CQC modal combination of the first three modes, it is possible to approximate maximum base shear, as shown in Equation 12. In order to provide generalized results and avoid the definition of a default story mass, the base shear is normalized by the total mass and denoted as $V_{b, norm}$, and linearized equations are developed for this new EDP, as shown in Equation 13. Assuming that $V_{b, norm}$ has a lognormal distribution (Equation 14), the mean $\mu_{\ln V_{b, norm}}$ and variance $\sigma_{\ln V_{b, norm}}^2$ are calculated according to

Equation 15, using the format previously discussed for $\ln \delta_k$ and for $\ln IDR_k$. In these equations, $V_i = (\sum_{s=1}^{NS} \Phi_{i,s})^2 / (NS \cdot \sum_{s=1}^{NS} \Phi_{i,s}^2)$, $VV_i = V_i e^{\mu_{\ln Sa,i}} \sum_{t=1}^3 V_t \alpha_{T_i, T_t} e^{\mu_{\ln Sa,t}}$, and $CQCV = \sum_{i=1}^3 \sum_{j=1}^3 V_i V_j \alpha_{T_i, T_j} e^{\mu_{\ln Sa,i}} e^{\mu_{\ln Sa,j}}$. These simple linearized formulae, paired with any GMM, will provide a quick estimate of the structural response to a selected ground motion scenario, and they can be easily added to Probabilistic Seismic Hazard calculation software for the direct generation of EDP hazard curves.

$$V_{bi} = V_{bi}^{st} S_a(T_i)$$

$$V_{bi}^{st} = \frac{(L_i^h)^2}{M_i} = \frac{\left(\sum_{t=1}^n m_t \Phi_{i,t}\right)^2}{\sum_{t=1}^n m_t \Phi_{i,t}^2} = m \frac{\left(\sum_{t=1}^n \Phi_{i,t}\right)^2}{\sum_{t=1}^n \Phi_{i,t}^2} \quad (11)$$

$$V_b = \sqrt{\sum_{i=1}^3 \sum_{j=1}^3 \left[V_{bi}^{st} S_a(T_i) \right] \alpha_{T_i, T_j} \left[V_{bj}^{st} S_a(T_j) \right]} \quad (12)$$

$$V_{b,norm} = \frac{V_b}{NS \cdot m} = \sqrt{\sum_{i=1}^3 \sum_{j=1}^3 \frac{\left(\sum_{s=1}^{NS} \Phi_{i,s}\right)^2}{NS \sum_{s=1}^{NS} \Phi_{i,s}^2} \frac{\left(\sum_{s=1}^{NS} \Phi_{j,s}\right)^2}{NS \sum_{s=1}^{NS} \Phi_{j,s}^2} \alpha_{T_i, T_j} e^{\ln Sa,i} e^{\ln Sa,j}}$$

$$= \sqrt{\sum_{i=1}^3 \sum_{j=1}^3 V_i V_j \alpha_{T_i, T_j} e^{\ln Sa,i} e^{\ln Sa,j}} \quad (13)$$

$$\ln V_{b,norm} = \frac{1}{2} \ln \sum_{i=1}^3 \sum_{j=1}^3 V_i V_j \alpha_{T_i, T_j} e^{\ln Sa,i} e^{\ln Sa,j} \quad (14)$$

$$\begin{cases} \mu_{\ln Vb,norm} = \frac{1}{2} \ln (CQCV) \\ \sigma_{\ln Vb,norm}^2 = \frac{1}{CQCV^2} \sum_{i=1}^3 \sum_{j=1}^3 VV_i VV_j \rho_{\ln Sa,i,j} \sigma_{\ln Sa,i} \sigma_{\ln Sa,j} \end{cases} \quad (15)$$

IMPLEMENTATION

Analytical structural models

The set of structural models used in this study represents a simplified description of the elastic response of multistory buildings. The frames are modeled using an equivalent structural system consisting of two cantilever beams connected by rigid horizontal links that equal their lateral deflections. The two cantilevers experience purely flexural and shear deformations, respectively. Additional information about the model can be found in [Miranda and Taghavi 2005](#). The models are entirely parameterized, and their behavior can be fully described using two parameters. The first parameter is $\alpha = H \sqrt{GA_0/EI_0}$, which indicates the contribution of shear deformations to the overall lateral deflection of the structure;

GA_0 and EI_0 represent the shear stiffness and the flexural stiffness, respectively, at the base of the structure; and H is the height of the building. The second parameter is the first mode period of the MDOF system denoted as T_1 . Miranda and Taghavi 2005 have developed closed-form solutions for the modal properties of these buildings, and they are reported here for the reader's convenience. These equations are based on the assumption of constant mass and stiffness distribution along the height and plan regularity of the structure. Equation 16 allows the calculation of the structure's mode shapes at each floor using the dimensionless parameter $x = h/H$. The eigenvectors are normalized so that $\phi_i(x = 1) = 1.0$. Equation 19 describes the distribution of the higher mode periods with respect to the first mode period T_1 . In these equations, i is the mode number, γ_i is the eigenvalue parameter and is calculated by finding the roots of Equation 17, and η_i is calculated through Equation 18. Figure 1 represents the variation of the first three mode shapes with α for a 20-story structure with a first period of 4 s. The red line represents $\alpha = 0$ (shear wall), the blue line represents $\alpha = 3.125$ (dual system), and the black line represents $\alpha = 12.5$ (moment-resisting frame).

$$\phi_i(x) = \frac{\sin \gamma_i x - \gamma_i (\alpha^2 + \gamma_i^2)^{-1/2} \sinh x \sqrt{\alpha^2 + \gamma_i^2} + \eta_i [\cosh x \sqrt{\alpha^2 + \gamma_i^2} - \cos \gamma_i x]}{\sin \gamma_i - \gamma_i (\alpha^2 + \gamma_i^2)^{-1/2} \sinh \sqrt{\alpha^2 + \gamma_i^2} + \eta_i [\cosh \sqrt{\alpha^2 + \gamma_i^2} - \cos \gamma_i]} \quad (16)$$

Figure 1. Influence of α on the first three mode shapes for a 20-story structure with $T_1 = 4$ s.

$$2 + \left[2 + \frac{\alpha^4}{\gamma_i^2 (\alpha^2 + \gamma_i^2)} \right] \cos \gamma_i \cosh \sqrt{\alpha^2 + \gamma_i^2} + \left[\frac{\alpha^2}{\gamma_i \sqrt{\alpha^2 + \gamma_i^2}} \right] \sin \gamma_i \sinh \sqrt{\alpha^2 + \gamma_i^2} = 0 \quad (17)$$

$$\eta_i = \frac{\gamma_i^2 \sin \gamma_i + \gamma_i \sqrt{\alpha^2 + \gamma_i^2} \sinh \sqrt{\alpha^2 + \gamma_i^2}}{\gamma_i^2 \cos \gamma_i + (\alpha^2 + \gamma_i^2) \cosh \sqrt{\alpha^2 + \gamma_i^2}} \quad (18)$$

$$\frac{T_i}{T_1} = \frac{\gamma_1}{\gamma_i} \sqrt{\frac{\alpha^2 + \gamma_1^2}{\alpha^2 + \gamma_i^2}} \quad (19)$$

A building matrix consisting of 24 frames is developed for this study using the following combinations of number of stories, first mode period, and α . Number of stories (NS) is set equal to 4, 8, 20, and 40. Two variation of first mode period T_1 are considered: $T_1 = 0.1 \times NS$ and $T_1 = 0.2 \times NS$. Parameter α is set to 0 to represent shear walls; $\alpha = 12.5$ to represent moment-resisting frames; and $\alpha = 3.125$ for dual systems, as recommended by [Taghavi and Miranda 2005](#). A standard story height of 12 ft is chosen to calculate the roof drift ratio (RDR) and maximum interstory drift ratio ($MIDR$), defined as the maximum interstory drift ratio value over the building height. The normalized base shear $V_{b,norm}$ is calculated according to Equation 15.

Variability in structural properties affects periods and mode shapes, which makes the inclusion of epistemic variability in the assessment of EDPs quite cumbersome. This uncertainty, however, can be simplified by assuming that aleatory and epistemic variabilities are independent from each other ([Baker and Cornell 2003](#)). Within this setting, the total variance in the estimation of EDP can be calculated as the sum of the aleatory term (Equations 8, 10, and 15) and an epistemic term representing variability in EDP due to uncertainty in structural properties.

For a given 5% damping ratio ξ , the period correlation parameter α_{ij} in Equation 4 for these structures is a function of α alone. For the values of α considered in this study, α_{ij} is approximately zero, which implies that the SRSS combination would be sufficiently accurate to estimate the response of this set of structures.

Characteristic Earthquake Scenarios

The Campbell-Bozorgnia 2014 Horizontal Ground Motion Model (denoted as CB14) developed as part of the NGA-West2 project is used to obtain the $S_{a,i}$ statistics at the first three periods of the structure. Several earthquake scenarios and site locations are considered based on characteristic fault geometry considerations. Table 1 contains the fault parameters recommended in Appendix C of CB14 for the analyzed magnitude values in strike-slip and reverse-fault scenarios.

The sensitivity of the structural response to site location and condition is assessed by using a range of rupture distances R_{RUP} between 1 and 100 km, and values of the shear wave velocity V_{S30} equal to 255, 360, 525, 760, and 1070 m/s, representative of the National Earthquake Hazards Reduction Program (NEHRP) site categories D, CD, C, BC, and B,

Table 1. Characteristic fault geometry for (a) strike-slip and (b) reverse rupture scenarios. Table adapted from [Campbell and Bozorgnia 2014](#)

M	Z_{BOT} [km]	W [km]	Z_{TOR} [km]	Z_{HYP} [km]
(a) Strike-slip faulting				
3.5	15	0.5119	7.1449	7.5626
4.5	15	1.6572	7.1449	8.2623
5.5	15	5.3653	4.2887	7.2779
6.5	15	14.1259	0.8741	8.8705
7.5	15	15	0.0	10.2267
8.0	15	15	0.0	10.2267
(b) Reverse faulting				
3.5	15	0.5119	7.3116	7.6374
4.5	15	1.6572	7.3116	8.3663
5.5	15	5.3653	7.3116	10.3008
6.5	15	16.0763	3.6324	11.6288
7.5	15	20.5595	0.4622	10.6889

respectively. The strike-slip scenarios are characterized by a 90-degree dip angle, and the reverse-fault scenarios are characterized by a 45-degree dip.

Sensitivity of EDPs to ground motion parameters

The response of the aforementioned structural models to several strike-slip and reverse faulting earthquake scenarios is investigated. The results, shown in the following figures, validate the linearized formulae obtained previously by comparing them with the median of EDPs obtained through modal analysis with CQC modal combination, Equations 6, 9, and 14, in which the spectral accelerations $S_a(T_i)$ are obtained through 1,000,000 Monte Carlo simulations whose means and covariance matrix match CB14. The correlation coefficients, $\rho_{\ln S_a,ij}$, are calculated using the equations provided by [Baker and Jayaram 2008](#). This formulation is based on the outdated NGA-West1 ground motion database and the [Chiou and Youngs 2008](#) GMM, but its results are still relevant and accurate, as recently verified by [Baker and Bradley 2017](#). Three EDPs are selected to investigate their sensitivity to earthquake parameters: maximum interstory drift ratio $MIDR$, roof drift ratio RDR , and normalized base shear $V_{b,norm}$. $MIDR$ is calculated as the maximum IDR_k among the stories (i.e., k) of the building, where IDR_k is a random variable obtained using the linearized formulae presented in Equation 10 and Monte Carlo simulations, as explained previously. RDR is calculated as the roof displacement δ_{NS} divided by the height of the building, where each story has a set height of 12 ft. As previously mentioned, $V_{b,norm}$ is based on the assumption of constant mass on all stories. A characteristic fault geometry is assigned to each investigated value of magnitude (see Table 1). All the median $MIDR$ and RDR results are presented as ratios, while the median $V_{b,norm}$ results are presented in terms of g . In all of the following plots, the continuous lines represent the median results of the linearized equations, and the dots are obtained through Monte Carlo simulations.

As will be seen in the following figures, the proposed equations provide a fairly accurate estimate of the median EDP, especially for lower values of V_{S30} and larger magnitudes. When considering only values of $MIDR$ larger than 0.5%, the maximum underestimation using the linearized equations when α is 0 is 14% for 40-story buildings and less than 10% for shorter structures. Larger values of α provide slightly less accurate estimates, with errors up to 17%. An even better agreement is seen for RDR , with a maximum underestimation of 7% for results of interest (0.5% or larger), 3.5% for strike-slip faults. The estimates for base shear are within 14% from the simulations for values larger than 0.3g (10% for strike-slip faults).

Figure 2 shows the variation of median $MIDR$ with R_{RUP} (closest distance to the fault rupture plane) for strike-slip scenarios with increasing values of V_{S30} for a sample 20-story shear-wall building ($\alpha = 0$) and first mode period $T_1 = 4$ s. The four lines represent different values of magnitude: $M = 4.5$ is in red, $M = 5.5$ is in blue, $M = 6.5$ is in black, and $M = 7.5$ is in green. Results are only shown for distances compatible with the fault geometry, that is, $R_{RUP} \geq Z_{TOR}$, where Z_{TOR} is the depth to the top of the fault (see Figure 14). Figure 2 shows the distance attenuation and magnitude scaling of $MIDR$. Moreover, the median values of $MIDR$ consistently decrease with increasing V_{S30} , even though this effect is secondary. Figure 3 presents the same results, where the median $MIDR$ is plotted against magnitude.

Figure 2. Variation of median $MIDR$ with distance R_{RUP} for strike-slip scenarios with different V_{S30} for a 20-story building with $T_1 = 4$ s and $\alpha = 0$. (a) $V_{S30} = 255$ m/s, (b) $V_{S30} = 360$ m/s, and (c) $V_{S30} = 760$ m/s.

Figure 3. Variation of median $MIDR$ with magnitude for strike-slip scenarios with different V_{S30} for a 20-story building with $T_1 = 4$ s and $\alpha = 0$. (a) $V_{S30} = 255$ m/s, (b) $V_{S30} = 360$ m/s, and (c) $V_{S30} = 760$ m/s.

Four values of R_{RUP} are represented: $R_{RUP} = 5$ km is in red, $R_{RUP} = 10$ km is in blue, $R_{RUP} = 30$ km is in black, and $R_{RUP} = 100$ km is in green. Figure 4 and Figure 5 present the variation of the median of RDR and the median of $V_{b,norm}$ with R_{RUP} for two sample shear-wall structures for strike-slip scenarios with $V_{S30} = 360$ m/s, using the same color convention of Figure 2. As previously noted, the linearized equations provide a close approximation to the simulated results.

The proposed linearized equations are capable of capturing the variance of EDPs with an acceptable accuracy. Figure 6 and Figure 7 present the variation of $\sigma_{\ln \delta}$ for the last story of the structure with magnitude for strike-slip scenarios with increasing V_{S30} for a four-story and a 40-story shear-wall building, respectively. Since $\ln RDR = \ln \delta_{NS} - \ln (NS \times h)$, $\sigma_{\ln \delta, NS}$ is equivalent to the standard deviation of $\ln RDR$, $\sigma_{\ln RDR}$. Results are shown for four values of distance, where $R_{RUP} = 5$ km is in red, $R_{RUP} = 10$ km is in blue, $R_{RUP} = 30$ km is in black, and $R_{RUP} = 100$ km is in green. Figures 6 and 7 show that $\sigma_{\ln \delta}$ depends mostly on earthquake magnitude, and that the effect of distance on this parameter is secondary and disappears on sites located on hard soil and for more flexible structures. These conclusions are compatible with the GMM variance formulation.

Figure 4. Variation of median RDR with distance R_{RUP} for strike-slip scenarios with $V_{S30} = 360$ m/s for two sample buildings. (a) $NS = 8$, $T_1 = 1.6$ s, $\alpha = 0$ and (b) $NS = 40$, $T_1 = 8$ s, $\alpha = 0$.

Figure 5. Variation of median $V_{b, norm}$ with distance R_{RUP} for strike-slip scenarios with $V_{S30} = 360$ m/s for two sample buildings. (a) $NS = 8$, $T_1 = 1.6$ s, $\alpha = 0$ and (b) $NS = 40$, $T_1 = 8$ s, $\alpha = 0$.

Figure 8 and Figure 9 show the variation of within-event standard deviation of $\ln MIDR$, denoted as $\phi_{\ln MIDR}$, and between-event standard deviation of $\ln MIDR$, denoted as $\tau_{\ln MIDR}$, for an eight-story and a 20-story shear-wall building respectively, for a strike-slip scenario with $V_{S30} = 360$ m/s. The pictures show the results for four values of magnitude: $M = 4.5$ is in red, $M = 5.5$ is in blue, $M = 6.5$ is in black, and $M = 7.5$ is in green. The figures show that $\tau_{\ln MIDR}$ is systematically lower than $\phi_{\ln MIDR}$. The total standard deviation of $\ln MIDR$, $\sigma_{\ln MIDR}$, is equal to the SRSS of the two, with a maximum difference of 1.3%. As previously noted, the EDP standard deviation shows a weak dependence on distance and shear wave velocity, and it is mostly sensitive to magnitude.

Figure 6. Variation of the last story $\sigma_{\ln \delta}$ with magnitude for strike-slip scenarios with different V_{S30} for a four-story building with $T_1 = 0.4$ s and $\alpha = 0$. (a) $V_{S30} = 255$ m/s and (b) $V_{S30} = 760$ m/s.

Figure 7. Variation of the last story $\sigma_{\ln \delta}$ with magnitude for strike-slip scenarios with different V_{S30} for a 40-story building with $T_1 = 4$ s and $\alpha = 0$. (a) $V_{S30} = 255$ m/s and (b) $V_{S30} = 760$ m/s.

Figure 10 presents the variation with distance of the base shear standard deviation $\sigma_{\ln V_{b,norm}}$ for a 20-story structure subjected to a strike-slip scenario with V_{S30} equal to 360 m/s and 760 m/s. The usual color convention is used to identify the four values of magnitude, using lines for the linearized results and dot markers for the Monte Carlo simulations. The figures show that the linearized equations systematically provide an accurate estimate of the result variance. It is also possible to conclude that the dispersion decreases for larger magnitudes but is not largely affected by the fault-to-site

Figure 8. Variation of ϕ_{InMIDR} , τ_{InMIDR} , and σ_{InMIDR} with distance R_{RUP} for strike-slip scenarios with $V_{S30} = 360$ m/s for an eight-story building with $T_1 = 0.8$ s and $\alpha = 0$. (a) ϕ (within-event), (b) τ (between-event), and (c) σ (total).

distance. Figure 11 presents the median *MIDR* and *RDR* results in terms of V_{S30} for a strike-slip scenario with $R_{RUP} = 10$ km for a 40-story shear-wall building with $T_1 = 8$ s. The results are presented for four values of magnitude, using the usual color convention. The previous observations are confirmed, and the median estimated EDP is systematically lower on harder soil.

The linearized equations are capable of capturing statistical parameters (i.e., mean and standard deviation) of EDPs in reverse faulting scenarios with an acceptable accuracy. Figure 12 shows the variation of median *MIDR* with distance for sites located on the hanging wall of a reverse fault subjected to characteristic earthquake scenarios with $V_{S30} = 360$ m/s. Two sample structures and the usual four values of magnitude are shown. In order to highlight the hanging-wall effect, Figure 13 presents the same results as a function of R_X , which is the surface projection of R_{RUP} . Positive values, or R_X , represent sites located on the hanging wall side of the fault, as shown in Figure 14. Figure 13 shows that the median EDP is larger on the hanging wall side, especially for large magnitudes, with the effect dissipating for sites more than 30 km away from the fault.

In order to further simplify the EDP formulation, one additional level of approximation is introduced and analyzed. Figure 15, Figure 16, and Figure 17 represent the contribution of the first mode alone to the total *MIDR*, *RDR*, and $V_{b,norm}$, respectively, for strike-slip

Figure 9. Variation of ϕ_{InMIDR} , τ_{InMIDR} , and σ_{InMIDR} with distance R_{RUP} for strike-slip scenarios with $V_{S30} = 360$ m/s for a 20-story building with $T_1 = 2$ s and $\alpha = 0$. (a) ϕ (within-event), (b) τ (between-event), and (c) σ (total).

Figure 10. Variation of $\sigma_{InVb, norm}$ with distance R_{RUP} for strike-slip scenarios with different V_{S30} for a 20-story building with $T_1 = 4$ s and $\alpha = 0$. (a) $V_{S30} = 255$ m/s and (b) $V_{S30} = 760$ m/s.

Figure 11. Variation of median $MIDR$ and RDR with shear wave velocity V_{s30} for strike-slip scenarios with $R_{RUP} = 10$ km for a 40-story building with $T_1 = 8$ s and $\alpha = 0$. (a) $MIDR$ and (b) RDR .

Figure 12. Variation of median $MIDR$ with distance R_{RUP} for reverse faulting scenarios with $V_{s30} = 360$ m/s for two sample structures. (a) $NS = 8$, $T_1 = 0.8$ s, $\alpha = 0$ and (b) $NS = 20$, $T_1 = 2$ s, $\alpha = 0$.

scenarios with $V_{s30} = 360$ m/s. Only the linearized results are shown for values of magnitude equal to 4.5 (red), 5.5 (blue), 6.5 (black), and 7.5 (green). Figure 15 shows that the first mode response adequately approximates the median $MIDR$ only for short structures, unless $M \geq 6.5$, and the approximation improves with lower V_{s30} and lower α . The level of approximation is much higher for RDR , with the first mode explaining more than 80% of the total EDP for magnitudes larger than 5.5. The first mode approximation for base shear is, generally, unsatisfactory. With the exception of short, stiff structures, the first mode fails to adequately represent the total response even for large magnitude events, highlighting the influence of higher modes on this EDP. These conclusions apply for reverse faulting scenarios as well.

Figure 13. Variation of median $MIDR$ with distance R_X for reverse faulting scenarios with $V_{S30} = 360$ m/sec for two sample structures. (a) $NS = 8$, $T_1 = 0.8$ s, $\alpha = 0$ and (b) $NS = 20$, $T_1 = 2$ s, $\alpha = 0$.

Figure 14. Two-dimensional (2-D) illustration of distance measures for multiple sites located on reverse faults, adapted from [Kaklamanos et al. 2011](#).

To conclude, Figure 18 shows the $MIDR$ between reverse faulting and strike-slip scenarios with the same ground motion characteristics. Even though the representative fault geometries for the two faulting scenarios are slightly different (see Table 1), the figures provide an approximate insight on the scaling due to reverse faulting and hanging-wall effects. The results are plotted using the linearized equations for the four aforementioned magnitude levels and show that the median $MIDR$ is up to 2.5 times larger for near-fault sites located on a hanging wall, where this factor increases for larger V_{S30} and shorter periods. The scaling peaks at distances R_{RUP} of approximately 10 km, progressively disappearing with larger distances.

Figure 15. First mode contribution to $MIDR$ for strike-slip scenarios with $V_{S30} = 360 \text{ m/s}$ for two sample structures. (a) $NS = 8$, $T_1 = 1.6 \text{ s}$, $\alpha = 0$ and (b) $NS = 20$, $T_1 = 4 \text{ s}$, $\alpha = 0$.

Figure 16. First mode contribution to RDR for strike-slip scenarios with $V_{S30} = 360 \text{ m/s}$ for two sample structures. (a) $NS = 20$, $T_1 = 4 \text{ s}$, $\alpha = 0$ and (b) $NS = 40$, $T_1 = 8 \text{ s}$, $\alpha = 0$.

Figure 17. First mode contribution to $V_{b, norm}$ for strike-slip scenarios with $V_{S30} = 360 \text{ m/s}$ for two sample structures. (a) $NS = 8$, $T_1 = 1.6 \text{ s}$, $\alpha = 0$ and (b) $NS = 40$, $T_1 = 4 \text{ s}$, $\alpha = 0$.

Figure 18. Ratio between median $MIDR$ from reverse faulting and from strike-slip scenarios with $V_{S30} = 760$ m/s for two sample structures. (a) $NS = 8$, $T_1 = 0.8$ s, $\alpha = 0$ and (b) $NS = 20$, $T_1 = 4$ s, $\alpha = 0$.

CONCLUSIONS

This paper introduces a simplified analytical model for assessing the response of structures as a function of earthquake parameters, denoted as PPEs. PPEs are conceptually novel and can be readily included in any hazard calculation program to directly estimate EDP hazard curves. Utilization of PPEs in performance-based seismic assessment eliminates one source of uncertainty in loss estimation by removing the intermediate step required to relate seismic hazard to ground motion intensity measure, and directly connects the response of a structure to seismic hazard. PPEs can also be utilized in the context of performance-based seismic design where seismic demands can be directly computed and compared with component and system capacity parameters in a probabilistic (i.e., reliability) format without the necessity of characterizing a ground motion intensity measure. The computational cost for generating PPEs is high compared with regular GMMs; development of PPEs for a given structure requires many response history analyses and a sophisticated probabilistic model to string vectors of earthquake parameters and building response parameters. The advancements in ordinary computational resources, such as personal computers, however, can make the development of PPEs a feasible task.

The PPEs presented herein utilize existing GMMs to obtain median and dispersion of EDPs with earthquake parameters. The proposed model for generating PPEs is relatively simple and utilizes existing knowledge in the characterization of building response, given ground motion spectral acceleration. The considered EDPs include floor displacement δ_k , interstory drift ratio IDR_k , and normalized base shear $V_{b,norm}$. These EDPs are not adequate for loss assessment of building structures. They are, however, a starting point for future and further development in generating more sophisticated PPEs complete enough for loss assessment. The simplified formulae are then used to develop the distributions of RDR , $MIDR = \max(\{IDR_k : k = 1, \dots, NS\})$, and $V_{b,norm}$. The proposed formulation is validated through the comparison with 1,000,000 Monte Carlo simulations.

The proposed formulation can be applied to any building structure with known periods and mode shapes. In this study, a shear-flexural beam characterized by the structural

parameter α and the first mode period T_1 was used. A sample application of the PPE formulation is as follows:

1. Select the building of interest. If using the shear-flexural beam model, identify T_1 and α , and use Equations 16 and 19 to calculate the mode shapes and the additional periods, respectively. If using a building model, identify the first n number of periods and mode shapes.
2. Use the mode shapes to calculate the modal participation factors.
3. Select the GMM to be used; select the earthquake scenario of interest, described by the ground motion parameters to be used in the GMM. If using CB14, select M , R_{RUP} , and V_{S30} , and use Table 1 for additional parameters.
4. Use the GMM to calculate the statistics of $\ln S_a$ for the first n number of periods of interest.
5. Use these values in Equations 8, 10, and 15 to find the probability distribution of $\ln \delta$, $\ln IDR$, and $\ln V_b$ for the selected earthquake scenario.

The distributions for EDPs (i.e., RDR , $MIDR$, and $V_{b,norm}$) show that the proposed PPEs provide an excellent approximation to the actual modal analysis results both in terms of median and dispersion. The trends in variation of EDPs obtained from the PPEs developed herein are compatible with the equivalent IM patterns published by the GMM authors (Campbell and Bozorgnia 2014), further confirming the validity of the proposed analytical formulation. The results of this paper highlight that the elastic structures used in this study are generally first mode dominant, and that the RDR could be adequately approximated by the first mode contribution alone. Meanwhile, the first mode approximation for $MIDR$ is inadequate for tall, flexible structures subject to ground motions with lower magnitudes. The first mode approximation is generally not valid for base shear, which is strongly influenced by higher modes at all magnitude levels.

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APPENDIX

Please refer to the online version of this article to access the supplementary material provided in the Appendix.

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