Validation of Ground-Motion Simulations through Simple Proxies for the Response of Engineered Systems

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Abstract We propose a list of simple parameters that act as proxies for the response of more complicated engineered systems and therefore can be studied to validate new methods of ground-motion simulation for engineering applications. The primary list of parameters includes correlation of spectral acceleration across periods, ratio of maximum-to-median spectral acceleration across all horizontal orientations, and the ratio of inelastic-to-elastic displacement, all of which have reliable empirical models against which simulations can be compared. We also describe several secondary parameters, such as directivity pulse period and structural collapse capacity, that do not have robust empirical models but are important for engineering analysis. We then demonstrate the application of these parameters to exemplify simulations computed using a variety of methods, including stochastic finite fault, Graves–Pitarka hybrid broadband, and a composite source model. In general, each simulation method matches empirical models for some parameters and not others, indicating that all relevant parameters need to be carefully validated.

Online Material: Tables of ground-motion records and simulations selected to have comparable response spectra, and MATLAB code to compute simple proxies for the response of engineering systems.

Introduction

Methods to simulate ground motions are rapidly changing and improving because of advances in geophysics and increases in computing power. There are many simulation methods, including many that are based on stochastic process (e.g., Jurkevics and Ulrych, 1978; Der Kiureghian and Crempien, 1989; Mobarakeh et al., 2002; Pousse et al., 2006), stochastic point source and finite fault (e.g., Boore, 1983; Beresnev and Atkinson, 1997; Motazedian and Atkinson, 2005), and hybrid broadband (e.g., Hartzell et al., 1999, 2005; Graves and Pitarka, 2010; Mai et al., 2010; Schmedes et al., 2010). As these methods continue to improve, engineers could benefit from using ground-motion simulations, particularly for infrequently observed conditions such as large magnitude and short distance events; however, before simulations can be used for engineering applications, validation is required to demonstrate that simulations have similar characteristics to real ground motions.

There have been many efforts to validate ground-motion simulations by comparing simulations of historical earthquakes to relevant recordings. These efforts focus on either the validation of simple ground-motion intensity measures, such as spectral acceleration and modified Mercalli intensity (Hartzell *et al.*, 1999, 2005; Aagaard *et al.*, 2008), or the structural response of idealized single-degree-of-freedom (SDOF) and multi-degree-of-freedom (MDOF) systems as a proxy for real structures (e.g., Bazzurro *et al.*, 2004; Iervolino *et al.*, 2010; Galasso *et al.*, 2012, 2013; Jayaram and Shome, 2012). Some general goodness-of-fit metrics also exist that measure the similarity between simulated and recorded time histories through a combination of parameters such as peak ground acceleration, peak ground velocity, spectral acceleration (SA) at multiple periods, and some structural response parameters, such as inelastic-to-elastic displacement ratios (S_{di}/S_{de}) (e.g., Anderson, 2004; Kristekova *et al.*, 2006; Olsen and Mayhew, 2010). However, because these previous validation efforts only compare simulations to historical recordings, they are not generalizable to simulations of future earthquake scenarios for which no recordings exist.

Some studies do compare simulations to empirical ground-motion prediction equations (GMPEs, previously known as attenuation relationships) (Frankel, 2009; Star *et al.*, 2011). But GMPEs are primarily based on observations of historical events and may be problematic when used to predict a situation that occurs very infrequently where the GMPE relies on extrapolation, like spectral acceleration amplitudes for large magnitudes and short distances.

Past validation efforts also tend to focus on a specific algorithm or set of simulations and do not explicitly propose

a procedure for validating future simulations. Because simulation methods are constantly changing, a consistent procedure for the engineering validation of simulations is essential, and the lack thereof leads to ambiguity in the selection of appropriate simulations.

We propose a validation framework that consists of simple parameters that act as proxies for the response of more complicated engineering models and have robust empirical models against which any simulation method can be validated, and we demonstrate the use of this framework for sample simulations of different methods from the Southern California Earthquake Center (SCEC) Broadband Platform (BBP) validation exercise (Dreger *et al.*, unpublished report, 2014; see Data and Resources). The main objective of the BBP validation exercise was to validate elastic spectral response, and the parameters proposed in this study are intended as a supplement, not a replacement, to this validation. The proposed parameters are also intended for ensembles of ground motions rather than individual ground motions.

Validation Framework

The proposed validation framework consists of four steps: (1) identify the application, (2) identify relevant ground-motion parameters and potential proxies, (3) compute proxies for simulations of interest, and (4) identify discrepancies and find potential causes. In this study, we consider building response and we identify primary and secondary parameters that are relevant to the seismic response of buildings. In the Example Study of Validation Framework section, we compute these parameters for ground-motion simulations from the SCEC BBP validation exercise, identify discrepancies between simulations, relevant recordings, and empirical models, and discuss potential causes.

The primary parameters considered here are correlation of spectral acceleration across periods, the ratio of maximumto-median spectral acceleration across all horizontal orientations, and the ratio of inelastic-to-elastic spectral displacement. The behavior of these parameters is well understood and robust to changes in earthquake characteristics such as rupture mechanism, magnitude, distance, and local site conditions. For each of these parameters, reliable empirical models exist that are not sensitive to extrapolation for infrequent events and can therefore be used as a baseline comparison against simulations for a very broad range of conditions.

The secondary parameters are directivity pulse periods and structural collapse capacity. These parameters are straightforward to compute and of engineering relevance but are less precisely constrained by an empirical model, so their use for validation requires more judgement.

The combination of these parameters is appropriate for the validation of building response, but the use of other relevant proxies may be important for other engineering applications. For example, significant duration (e.g., Kempton and Stewart, 2006) is important for many geotechnical applications, such as landslide analysis, and spatial correlation of intensity measures (e.g., Jayaram *et al.*, 2010) is important for distributed systems such as electrical grids and water distribution networks. Also, appropriate proxies may not exist for some causes of severe building damage, such as basin effects and fling step, due to lack of observations.

Correlation of ε across Multiple Periods

The parameter ε accounts for the wide variation in ground-motion intensity at sites from earthquakes with similar rupture mechanism, magnitude, and distance. ε is defined as the normalized difference between an observed spectral acceleration and the mean predicted natural log of spectral acceleration from a GMPE (e.g., Abrahamson *et al.*, 2008):

$$\varepsilon(T) = \frac{\ln SA(T) - \mu_{\ln SA}(M, R, \Theta, T) - \eta \tau_{\ln SA}}{\phi_{\ln SA}}, \quad (1)$$

in which $\ln SA(T)$ is the natural log of the observed spectral acceleration at period T, $\mu_{\ln SA}(M, R, \Theta, T)$ is the predicted mean of the natural log of spectral acceleration from a GMPE with magnitude M, distance R, and other parameters Θ (such as local site condition), $\phi_{\ln SA}$ and $\tau_{\ln SA}$ are the within-event and between-event standard deviations from a GMPE, respectively, and η is the between-event term. The within-event and between-event variabilities represent the record-to-record and earthquake-to-earthquake variabilities, respectively, and are zero-mean, independent, normally distributed random variables (Al Atik *et al.*, 2010). Because of the normalization by $\mu_{\ln SA}$ and $\phi_{\ln SA}$, ε is well represented by a standard normal distribution with a mean of zero and a standard deviation of one. An example of ε versus period is shown in Figure 1 for a recording from the 1994 Northridge earthquake.

The value of ε depends on the period T, and ε at different periods are correlated in a predictable way. Even though the process of computing ε relies on an empirical model for ground-motion amplitude, the resulting correlation is very stable for varying reference GMPEs and across earthquake records of all tectonic regimes, rupture mechanisms, magnitudes, distances, and local site conditions. For example, correlations computed from shallow crustal events (Baker and Jayaram, 2008) and subduction zone events (Al Atik, 2011) are extremely similar (Fig. 2).

The correlation coefficient of ε at two periods is computed as

$$\rho_{\varepsilon(T_1),\varepsilon(T_2)} = \frac{\sum_{i=1}^{n} [\varepsilon_i(T_1) - \overline{\varepsilon(T_1)}] [\varepsilon_i(T_2) - \overline{\varepsilon(T_2)}]}{\sqrt{\sum_{i=1}^{n} [\varepsilon_i(T_1) - \overline{\varepsilon(T_1)}]^2 \sum_{i=1}^{n} [\varepsilon_i(T_2) - \overline{\varepsilon(T_2)}]^2}},$$
(2)

in which $\varepsilon_i(T_1)$ and $\varepsilon_i(T_2)$ are the *i*th observation of $\varepsilon(T_1)$ and $\varepsilon(T_2)$, $\varepsilon(T_1)$ and $\varepsilon(T_2)$ are the sample means of $\varepsilon(T_1)$ and $\varepsilon(T_2)$, respectively, and *n* is the number of observations (i.e., ground motions). The correlation of ε at multiple periods is an important proxy for demands on MDOF



Figure 1. (a) Response spectrum from the Pacific Palisades–Sunset station in the fault-normal direction for the 1994 Northridge earthquake, shown with the predicted median $\exp(\mu_{\ln SA})$ and total standard deviation $\sqrt{\phi_{\ln SA}^2 + \tau_{\ln SA}^2}$ from a ground-motion prediction equation (GMPE) and (b) ε from the same recording.



Figure 2. Correlation coefficient of ε between periods T_1 and T_2 , computed for shallow crustal and subduction zone earthquakes.

structures, which are excited at several periods. Therefore, we suggest that the correlation structure of ground-motion simulations should match those of an empirical model, such as Baker and Jayaram (2008).

Ratio of Maximum-to-Median Response across Orientations

The spectral acceleration of a multicomponent ground motion depends on the orientation of interest, and variation of spectral acceleration with orientation is captured by the parameters $SA_{RotD100}$ and SA_{RotD50} . For a multicomponent ground motion, a spectral acceleration value can be computed for the shaking in any horizontal direction. For a given period, spectral accelerations are computed for rotation angles from 0° to 180° (because 180°–360° are redundant), and



Figure 3. Displacement trace and spectral acceleration for an elastic oscillator with T = 5 s in all orientations, computed using the ground motions from the (a) Los Angeles–Saturn Street station with SA *RotD100* /SA*RotD50* \approx 1 and (b) Glendora–N Oakbank station with SA *RotD100* /SA*RotD50* \approx 1.41 from the 1994 Northridge earth-quake.

 SA_{RotDnn} is the *n*th percentile of the results. Thus $SA_{RotD100}$ is the maximum and SA_{RotD50} is the median of the spectral accelerations over all unique horizontal orientations (Boore *et al.*, 2006; Boore, 2010):

$$SA_{RotD50}(T) = medianSA(T, \theta)$$
 (3)

and

$$SA_{RotD100}(T) = \max_{\alpha} SA(T, \theta),$$
 (4)

in which *T* is the period and θ is the orientation. SA_{*RotD*100} and SA_{*RotD*50} are computed independently at each period, so there is no single maximum or median orientation for a given ground motion.

The ratio of $SA_{RotD100}$ to SA_{RotD50} is a proxy for the polarization of a ground motion. If $SA_{RotD100}$ is approximately equal to SA_{RotD50} (i.e., $SA_{RotD100}/SA_{RotD50} \approx 1$), then the structural response is about the same in all orientations (Fig. 3a). If $SA_{RotD100}$ is much larger than SA_{RotD50} (i.e., $SA_{RotD100}/SA_{RotD50} \approx \sqrt{2} = 1.41$, which is the maximum



Figure 4. Empirical models for the geometric mean ratio of $SA_{RotD100}$ to SA_{RotD50} . The Beyer and Bommer empirical model reports $SA_{RotD100}/SA_{GMRotJ50}$, in which $SA_{GMRotJ50}$ is the median of the geometric mean of spectral accelerations in a direction based on a single period (Boore *et al.*, 2006) but is corrected to $SA_{RotD100}/SA_{RotD50}$ here using Boore (2010).

possible ratio), then the structural response is polarized in one orientation (Fig. 3b).

The median $SA_{RotD100}/SA_{RotD50}$ ratio from a suite of ground-motion recordings has a very stable relationship with period and is not dependent on magnitude, distance, or local site conditions. This ratio is important for predictions of structural behavior, particularly for 3D structural models, which respond in all orientations. Therefore, simulations should have ratios consistent with empirical models (e.g., Beyer and Bommer, 2006; Shahi and Baker, 2013) (Fig. 4).

Ratio of Inelastic-to-Elastic Displacement

Engineers typically design a structure assuming that it will behave inelastically during a large earthquake, and therefore the inelastic relative to elastic behavior of an SDOF oscillator is an important proxy for the behavior of real structures. One common way to quantify the difference between inelastic and elastic structural behavior is the ratio of the displacement of a bilinear inelastic oscillator S_{di} , to an elastic oscillator S_{de} (see Fig. 5 for force–displacement curves). This ratio depends on the period T and the strength reduction factor R_{μ} , which is the force required for the oscillator to remain elastic (F_e) over the yield force F_y or, equivalently, S_{de} over the yield displacement d_y (Chopra, 2001).

The dependence of $S_{\rm di}/S_{\rm de}$ on *T* and R_{μ} is well understood from empirical data. In general, the mean ratio is greater than one at short periods and increases with R_{μ} , and it is close to one at periods around 1 s where the equal displacement rule applies (Veletsos and Newmark, 1960). In order for ground-motion simulations with a given elastic spectrum to provide a reliable estimate of inelastic structural behavior, their inelastic-to-elastic displacement ratio should match empirical models, such as Tothong and Cornell (2006), which assumes a bilinear oscillator with $\alpha = 0.05$ and only depends on earthquake magnitude in addition to *T* and R_{μ} (Fig. 6). Instead of using the true R_{μ} , which depends



Figure 5. Force–displacement behavior of a bilinear inelastic and an elastic oscillator, in which K is the elastic stiffness (which relates to the period T), α is the strain hardening ratio, F_y is the yield force, F_e is the elastic force, d_y is the yield displacement, S_{de} is the displacement of the elastic oscillator, S_{di} is the displacement of the inelastic oscillator, and R_u is the strength reduction factor.

on S_{de} and therefore cannot be known before an earthquake occurs, Tothong and Cornell (2006) use the predicted median reduction factor, $\hat{R}_{\mu} = \hat{S}_{de}/d_y$, in which \hat{S}_{de} is the median prediction from a GMPE.

Some empirical models provide a direct prediction of S_{di} at a specific R_{μ} , such as Bozorgnia *et al.* (2010), but depend on many parameters (e.g., rupture mechanism, magnitude, distance, local site conditions, etc.) and may or may not extrapolate well to infrequent events such as large magnitudes and short distances. In contrast, the ratio of inelastic-to-elastic displacement is more stable with changes in earthquake and site conditions than S_{di} alone (Miranda, 2000). Also, in engineering applications the elastic spectrum is often specified by the design procedure, so the behavior of an inelastic oscillator relative to an elastic one is a more representative proxy for typical engineering analysis. Therefore, we use Tothong and Cornell (2006), which predict the mean ratio of S_{di}/S_{de} , for later comparisons.

Other Ground-Motion Parameters

Other properties, such as directivity pulses and collapse capacity, are important for engineering applications but are not well constrained by empirical data and are more sensitive to changes in earthquake scenario and structural parameters. Therefore, these properties are not considered definitive for validation but are described and applied to example groundmotion simulations.

Directivity Pulses. Directivity pulses are a double-sided velocity pulse caused by constructive interference of seismic waves as a rupture propagates along a fault. They tend to occur at sites that are far from the epicenter, but close to the fault, and are strongest in the fault-normal direction. These pulses amplify structural response at long periods and are



Figure 6. Empirical model for the ratio of the displacement of a bilinear inelastic to an elastic oscillator S_{di}/S_{de} for an earthquake with magnitude (a) 6 and (b) 7, as a function of period *T* and predicted median reduction factor $\hat{R}_{\mu} = \hat{S}_{de}/d_y$, in which \hat{S}_{de} is the median prediction from a GMPE (Tothong and Cornell, 2006).

thus a serious design concern for structures located close to a fault (Somerville *et al.*, 1997; Somerville, 2003).

Because theory suggests that pulse period is closely related to the rise time of slip on the fault and the logarithm of rise time is proportional to magnitude (Somerville, 1998; Somerville et al., 1999), empirical models for pulse period typically follow a lognormal distribution for which the mean values are a function of earthquake magnitude (e.g., Bray and Rodriguez-Marek, 2004; Shahi and Baker, 2011). These empirical models are based on historical ground-motion recordings, from which pulse periods can be estimated using wavelet analysis (e.g., Baker, 2007), zero crossings (e.g., Bray and Rodriguez-Marek, 2004), or velocity spectra (e.g., Alavi and Krawinkler, 2001). However, because there are few recordings of large magnitude earthquakes at the short distances where directivity is most likely to occur, it is difficult to claim that these models provide a complete description of directivity pulses. In fact, ground-motion simulations may be able to provide a more accurate characterization of directivity pulses and near-fault ground motions because they capture finite-fault geometry and source-to-site path characteristics. But some cases may exist where it is useful to compare the pulse characteristics of simulations to empirical models.

Structural Collapse Capacity. The collapse capacity of a structure is the ground-motion intensity level that causes it to lose stability and collapse. One method to compute collapse capacity is an incremental dynamic analysis (IDA), and the final result is a collapse fragility curve that defines the probability of collapse given a ground-motion intensity (Ibarra and Krawinkler, 2005). An IDA is performed by incrementally scaling up a single ground motion until collapse occurs. The IDA is repeated for a set of ground motions to get a probabilistic description of collapse and is typically performed using the full nonlinear MDOF model of a structure of interest.

Because collapse capacity is highly dependent on structural properties such as period, peak-to-yield-displacement



Figure 7. Force–drift backbone curve for the proposed structural system, in which *T* is period, *H* is height, *M* is mass, *W* is weight, F_y is yield force, F_c is peak force, α_c is postpeak stiffness, d_c is displacement at peak force, d_y is yield displacement, and *K* is stiffness.

ratio, postpeak stiffness, and collapse mechanism (Zareian et al., 2010), it is a difficult parameter to validate. Therefore, we propose the use of three generic nonlinear SDOF models to act as a proxy for more complicated MDOF models. The proposed SDOFs have periods of T = 0.3, 0.8, and 2 s butthe same relative force-drift behavior, and they are representative of midrise concrete frame buildings (Fig. 7, in which $F_v/W = 0.198$, $F_c/W = 0.216$, $\alpha_c = -0.1$, and $d_c/d_v = 4$). Their hysteretic behavior is represented by a peak-oriented model with negligible cyclic deterioration (e.g., Ibarra and Krawinkler, 2005). For ground motions, we suggest using an existing set of recordings, then selecting a set of simulations with comparable elastic response spectra. The spectral equivalence controls for differences in elastic response, so discrepancies in the resulting collapse capacity must be due to other, less obvious properties of the ground motions. The collapse capacity can be computed for each



Figure 8. (a) Station locations for recorded and simulated ground motions, and (b) SA_{RotD50} spectra for recorded and simulated ground motions of the 1989 Loma Prieta earthquake. The SA_{RotD50} spectra are not scaled, resulting in large variability due to different distances, local site conditions, etc. The color version of this figure is available only in the electronic edition.

SDOF using each set of ground motions, and the collapse capacities for recordings can be compared to simulations.

Structural collapse capacity is essential for capturing the highly nonlinear behavior of structures, which plays a large role in seismic risk assessment, but collapse capacity can be difficult to compute and is highly dependent on structural modeling choices and ground-motion selection. These challenges make it a complicated and possibly less robust validation metric for ground-motion simulations, but certainly not any less important.

Example Study of Validation Framework

This section presents examples of the proposed parameters evaluated using sample ground-motion simulations. For each parameter, we attempt to show an example that matches the empirical model and an example that deviates. Because many were implemented without these checks in mind, the intent of this section is not to judge the simulations, but rather to demonstrate the computation of these parameters and discuss possible outcomes. Some of the simulation methods may be easily improved in future implementations on the SCEC BBP.

Example of Ground-Motion Simulations

The ground-motion simulations used in this paper were either computed by the authors using the SCEC BBP or as part of the SCEC BBP validation exercise (Dreger *et al.*, unpublished report, 2014; see Data and Resources), which includes 50 realizations of historical and nonhistorical events with varying slip distributions and using several ground-motion simulation methods. We obtained simulations of the 1989 Loma Prieta and 1994 Northridge earthquakes at 40 stations on rock site conditions ($V_{S30} = 863$ m/s). We obtained simulations using the stochastic finite fault (EXSIM) (Motazedian and Atkinson, 2005) and composite source model (CSM) (Zeng *et al.*, 1994) methods for Northridge and Loma Prieta, and the Graves–Pitarka hybrid broadband (GP) method (Graves and Pitarka, 2010) for Loma Prieta only (Figs. 8 and 9). For each simulation type, we chose the realization with the smallest average error between the elastic response spectra of recordings and simulations. (For Loma Prieta, we used realization number 10000044 of EXSIM, realization number 10000005 of GP, and realization number 10000040 of CSM; for Northridge, we used realization number 10000044 of EXSIM and realization number 10000047 of CSM.) For demonstration purposes, we chose to use historical rather than nonhistorical simulations because historical simulations can be compared with both empirical models and relevant recordings.

The shape of the elastic response spectra has a significant effect on the inelastic-to-elastic displacement ratio and the collapse capacity. To ensure that any observed differences were not simply due to differences in elastic spectra, we also used several sets of simulations that were selected based on matching their mean spectral shape to a similar set of recordings. For recordings, we used set 2 from a Pacific Earthquake Engineering Research Center (PEER) study (Baker et al., 2011; hereafter referred to as the PEER records), which consists of 40 recordings having elastic response spectra comparable with those expected from events with magnitudes close to 7 and distances close to 10 km (E) Table S1 available in the electronic supplement to this paper). For simulations, we obtained three realizations of the 1989 Loma Prieta earthquake using the CSM and GP methods, resulting in 120 ground motions for each method, and two realizations of Loma Prieta using the EXSIM method, resulting in 80 ground motions. (For EXSIM, we obtained realization numbers 10000044 and 10000014; for GP, we obtained realization numbers 10000005, 10000007, and 10000030; and for CSM, we obtained realization numbers 10000040,



Figure 9. (a) Station locations for recorded and simulated ground motions, and (b) SA_{RotD50} spectra for recorded and simulated ground motions of the 1994 Northridge earthquake. The SA_{RotD50} spectra are not scaled, resulting in large variability due to different distances, local site conditions, etc. The color version of this figure is available only in the electronic edition.



Figure 10. Elastic response spectra scaled at T = 0.8 s for set 2 from a Pacific Earthquake Engineering Research Center (PEER) study (Baker *et al.*, 2011; hereafter referred to as the PEER records) and several sets from different simulation methods selected based on their match with the mean spectral shape of the PEER records. The color version of this figure is available only in the electronic edition.

10000017, and 10000038.) From those 120 or 80, we selected 40 that best matched the mean of scaled spectra of the PEER records (Fig. 10 and E) Tables S2–S4). The mean spectrum of the selected simulations reasonably matches the mean spectrum of recordings scaled at several periods (Fig. 11), but the standard deviation varies (Fig. 12). For future validation purposes, we suggest using any set of recordings as long as the mean of scaled elastic spectra matches the simulations that are being validated.

Finally, we use hybrid broadband simulations from two sets of earthquake scenarios computed using SCEC BBP version 11.2.3, with the GP method for the rupture generator and low-frequency seismogram, and the San Diego State University method (Mai *et al.*, 2010) for high-frequency seismogram and site response. The first set contains 10 realizations of a magnitude 7 earthquake on the Hayward fault with 12 stations, each located 1 km from the fault with $V_{S30} = 500$ m/s (Fig. 13). The second set contains 10 realizations of a magnitude 7 earthquake on the San Andreas fault, with similar rupture geometry to the 1989 Loma Prieta earthquake, and 12 stations located 1 km from the surface projection of the fault with $V_{S30} = 500$ m/s (Fig. 14). Each set of scenarios contains 120 near-fault ground-motion simulations and is used for the evaluation of pulse periods.

Correlation of ε across Multiple Periods

Here we compare the statistics of ε from empirical models to recordings and simulations of the 1989 Loma Prieta earthquake. In this example, the between-event term in equation (1) is constant for a given period and cancels out because all ground motions are from the same event. The standard deviation of ε is expected to equal one at all periods because of the normalization in equation (1). The recordings and CSM simulations have a standard deviation close to one at all periods, the EXSIM simulations have a small standard deviation at all periods (≈ 0.24 to 0.64), and the GP simulations have a small standard deviation at periods shorter than 1 s (≈ 0.36 to 0.64) but close to 1 at longer periods (Fig. 15). At short periods, the EXSIM and GP simulation methods are very similar, resulting in similar behavior of ε . For GP simulations, the high- and low-frequency seismograms are spliced together at 1 s, so the change in standard deviation at this period suggests a lack of variability in the high-frequency simulation process, previously observed by Seyhan et al. (2013).

Correlations of ε at select periods are shown in Figure 16. In general, the recordings follow the empirical models whereas the simulations do not. Even the correlation of ε for the CSM simulations differs from empirical models, despite having a reasonable standard deviation of ε . The GP simulations match the empirical model between periods around 1 s and less than 0.1 s and between periods longer than 1 s where



Figure 11. Elastic response spectra for the PEER records and matched simulations, scaled at (a) T = 0.3 s, (b) T = 0.8 s, and (c) T = 3 s. The mean spectra match reasonably well except the CSM simulations at long periods. The color version of this figure is available only in the electronic edition.

the seismogram is deterministically computed. However, the correlation tends to attenuate too quickly with period and is low between periods close to 1 s, indicating that correlation may be lost when the high- and low-frequency seismograms are spliced together. The correlation of EXSIM simulations also attenuates too quickly with period, as expected because these simulations are based on white noise. This behavior was also observed previously for stochastic point-source simulations (Tothong and Cornell, 2007). The CSM simulations have high correlations at all periods, whereas the EXSIM simulations have low correlations at all periods relative to the empirical model and recordings.

Ratio of Maximum-to-Median Response across Orientations

The median $SA_{RotD100}$ to SA_{RotD50} ratios from recordings and simulations of the 1989 Loma Prieta earthquake are compared to empirical models (Fig. 17). EXSIM simulations are excluded because they only produce ground motions in a single orientation, so SA_{RotD50} and $SA_{RotD100}$ cannot be computed. The recordings match the empirical models reasonably well at all periods, and the GP simulations also match the empirical models well, except at periods between 1 and 4 s. The CSM simulations have a very large median ratio at periods shorter than 1 s (\approx 1.35 to 1.41), which is effectively 50% larger than comparable recordings (which have ratios ≈ 1.16 to 1.30) because the ratio can only have values between 1 and 1.41. This large ratio indicates that structural response to the CSM simulations is more polarized than expected. The polarization is especially strong at short periods because the currently implemented CSM method uses nonrandom radiation patterns at high frequencies, though this could be adjusted in a future implementation of the method on the SCEC BBP (John Anderson, personal comm., 2013).

The relationship between SA_{RotD100}/SA_{RotD50} and structural response can be seen in the displacement trace of an elastic oscillator. Using the ground motion with the median SA_{RotD100}/SA_{RotD50} from each ground-motion set, the displacement was calculated for an elastic oscillator with T = 0.2 s and T = 3 s in all orientations (Fig. 18). The elastic oscillator with T = 0.2 s responds to the CSM simulation with more polarization than the recording and GP simulation because the median SA_{RotD100}/SA_{RotD50} is much larger. Similarly, the elastic oscillator with T = 3 s responds to both the CSM and GP simulation with more polarization than the recording.

Ratio of Inelastic-to-Elastic Displacement

The ratio of inelastic-to-elastic displacement, S_{di}/S_{de} , for recordings and simulations of the 1989 Loma Prieta



Figure 12. Standard deviation of the natural log of the scaled elastic response spectra for the PEER records and matched simulations, scaled at (a) T = 0.3 s, (b) T = 0.8 s, and (c) T = 3 s. The color version of this figure is available only in the electronic edition.



Figure 13. The set of Hayward scenario simulations: (a) station locations and (b) response spectra in the north–south direction. This set contains 10 realizations of a magnitude 7 earthquake, with 12 stations located 1 km from the fault.

earthquake is compared to an empirical model at multiple periods (Fig. 19). At T = 1.6 s, the recordings and all simulations match the empirical model, but at T = 0.8 s, GP and CSM simulations have a mean ratio that is about 11% and 5%, respectively, larger than the empirical model. The inelastic behavior of an SDOF oscillator depends strongly on the

elastic response at longer periods because the effective natural period lengthens as the SDOF behaves nonlinearly (Iwan, 1980). Therefore, the discrepancy in S_{di}/S_{de} at different periods can be at least partially explained by the difference in relative shape of the scaled response spectra (Fig. 20). Scaled at T = 1.6 s, the mean response spectrum



Figure 14. The set of San Andreas scenario simulations: (a) station locations and (b) response spectra in the north–south direction. This set contains 10 realizations of a magnitude 7 earthquake, with 12 stations located 1 km from the surface projection of the rupture.



Figure 15. Standard deviation of ε for the recordings and simulations of the 1989 Loma Prieta earthquake, shown with the expected standard deviation of one. The color version of this figure is available only in the electronic edition.

is similar for the recordings and all simulations at periods slightly longer than 1.6 s; however, scaled at T = 0.8 s, the mean of GP and CSM simulations is larger than the recordings at longer periods, leading to a larger inelastic displacement and inelastic-to-elastic displacement ratio.

To control for the spectral shape, we compare the inelastic-to-elastic displacement ratio of the PEER records and matched simulations to an empirical model at multiple periods (Fig. 21). Now that the mean spectral shape is similar, S_{di}/S_{de} for the PEER records and all matched simulations at T = 0.8 s agrees with the empirical model. Yet discrepancies remain at T = 0.3 s and T = 3 s; for example, the ratio of EXSIM simulations is about 17% and 6% (respectively,) below recordings on average, even though their mean spectra are similar (see the mean spectra in Fig. 11). Also, at T = 0.3 s, the ratio of GP simulations is about 9% below recordings on average, even though the mean spectrum is similar. These discrepancies may be due to differences in the standard deviation of the scaled response spectra (recall Fig. 12). At T = 0.3 s, both the EXSIM and GP simulations have standard deviations smaller than recordings at periods longer than 0.3 s, leading to smaller S_{di}/S_{de} . At T = 3 s, the EXSIM simulations have a smaller standard deviation, whereas the GP simulations have a standard deviation that matches the PEER records, leading to a small S_{di}/S_{de} for the EXSIM simulations and an S_{di}/S_{de} that matches the PEER records for the GP simulations.

Other Ground-Motion Parameters

Directivity Pulses. Here we compare the distribution of pulse periods from the sets of scenarios on the Hayward and San Andreas faults with two empirical models (Bray and Rodriguez-Marek, 2004; Shahi and Baker, 2011). Pulse periods were extracted from the east–west component of all ground motions using wavelet analysis (Baker, 2007). Even though all earthquake scenarios are magnitude 7 with stations located 1 km from the fault, the San Andreas simulations are dominated by pulse periods between 1 and 2 s, whereas the Hayward simulations are dominated by longer pulse periods between 4 and 5 s (Fig. 22). The distribution of pulse periods from the San Andreas scenario is more similar to the empirical models than the Hayward scenario.

In general, the validation of pulse characteristics against empirical models is often not feasible because it requires a large sample of pulses—and therefore an even larger sample of ground-motion simulations. Because many simulation applications do not produce a large sample of near-fault ground motions, such as simulations of historical events, it can be difficult to statistically analyze their pulse properties. However, for the validation of historical simulations, we can directly compare the location and pulse periods that occurred in the actual event to the simulation of the event. Because Northridge contains more pulse-like ground motions than Loma Prieta (using wavelet analysis; e.g., Baker, 2007), we compare recordings to simulations in the north–south



Figure 16. Correlation of ε for recordings and simulations of the 1989 Loma Prieta earthquake, shown with an empirical model for shallow crustal earthquakes (Baker and Jayaram, 2008) and results from subduction zone earthquakes (Al Atik, 2011) at (a) $T_2 = 0.2$ s, (b) $T_2 = 0.8$ s, (c) $T_2 = 1.2$ s, and (d) $T_2 = 3$ s. The color version of this figure is available only in the electronic edition.



Figure 17. Ratio of $SA_{RotD100}$ to SA_{RotD50} from empirical models and the median ratio from recordings and simulations of the 1989 Loma Prieta earthquake. The color version of this figure is available only in the electronic edition.

direction of the Northridge earthquake. The recordings contain seven pulses, CSM simulations contain six pulses, and the EXSIM simulations contain only one pulse. The location of pulses tends to be in the northeast corner of the fault projection for the recordings and both sets of simulations (Fig. 23). The pulse periods for the CSM simulations are similar to recordings and range from 1 to 2.5 s, and the only pulse period for the EXSIM simulations is 2 s (Fig. 24). Results from GP simulations are omitted here because of space constraints.

Structural Collapse Capacity. We computed the collapse capacity using an IDA of the three proposed SDOF structures using the PEER records and matched simulations (Fig. 25). Because the mean and standard deviation of the elastic spectra scaled at T = 0.8 s are similar for all ground motions, the median collapse capacity of the SDOF with T = 0.8 s is also similar for all ground motions (all simulations are within 6% of the recordings). In contrast, the standard deviation of the elastic spectra scaled at T = 0.3 s is small for GP simulations, leading to a 12% increase in the median collapse capacity for the SDOF with T = 0.3 s, which is an unconservative estimate. Similarly, the standard deviation of the EXSIM elastic spectra scaled at T = 2 s is small, again leading to a 12% increase in the median collapse capacity for the SDOF with T = 2 s.

The results shown in Figure 25 are empirical cumulative distribution functions (not fragility functions) based on 40



Figure 18. Displacement traces of elastic oscillators subject to the ground motion with median $SA_{RotD100}/SA_{RotD50}$ ratio from Figure 17. Each ground motion is rotated so its $SA_{RotD100}$ occurs in the 0° orientation. The following ground-motion type, elastic period, and $SA_{RotD100}/SA_{RotD50}$ are shown in each subfigure: (a) recording, T = 0.2 s, $SA_{RotD100}/SA_{RotD50} = 1.2$; (b) GP simulation, T = 0.2 s, $SA_{RotD100}/SA_{RotD50} = 1.2$; (c) CSM simulation, T = 0.2 s, $SA_{RotD100}/SA_{RotD50} = 1.37$; (d) recording, T = 3 s, $SA_{RotD100}/SA_{RotD50} = 1.28$; (e) GP simulation, T = 3 s, $SA_{RotD100}/SA_{RotD50} = 1.37$; and (f) CSM simulation, T = 3 s, $SA_{RotD100}/SA_{RotD50} = 1.38$.



Figure 19. Inelastic-to-elastic displacement ratio S_{di}/S_{de} for recordings and simulations of the 1989 Loma Prieta earthquake as a function of \hat{R}_{μ} at (a) T = 0.8 s and (b) T = 1.6 s. The color version of this figure is available only in the electronic edition.

ground motions in each set, and there were no observations of collapse at spectral accelerations smaller than 0.3g. In contrast, a theoretical fragility function would have some finite probability of collapse at small spectral accelerations.

Conclusions

We proposed a validation framework for ground-motion simulations that largely consists of simple parameters that act



Figure 20. Elastic response spectra for recordings and simulations of the 1989 Loma Prieta earthquake scaled at (a) T = 0.8 s and (b) T = 1.6 s. The color version of this figure is available only in the electronic edition.



Figure 21. Inelastic-to-elastic displacement ratio S_{di}/S_{de} for PEER records and matched simulations as a function of \hat{R}_{μ} at (a) T = 0.3 s, (b) T = 0.8 s, and (c) T = 3.0 s. The color version of this figure is available only in the electronic edition.



Figure 22. Histogram of pulse period T_p in the east–west direction for the Hayward and San Andreas scenario simulations compared with two empirical models.

as proxies for the response of more complicated engineered systems and have robust empirical models that are insensitive to changes in earthquake scenario. The validation framework consisted of four steps: (1) identify application, (2) identify proxies, (3) compute proxies, and (4) identify discrepancies and potential causes. We also provided an example of the framework for building response using several different simulation methods from the SCEC BBP, including EXSIM, the GP method, and CSM.

First, we discussed the correlation of ε across periods, the ratio of SA_{*RotD*100} to SA_{*RotD*50}, and the ratio of inelasticto-elastic displacement because these parameters have robust empirical models against which simulations from a broad range of conditions can be compared. Correlation of ε captures the relative spectral response at different periods, making it an important proxy for demands on MDOF structures. From the example simulations used in this study, we saw that CSM simulations overestimated the correlation, whereas EXSIM simulations underestimated it. For GP simulations,



Figure 23. Map of pulse locations in the north-south direction during the 1994 Northridge earthquake for (a) recordings, (b) CSM simulations, and (c) EXSIM simulations.



Figure 24. Histogram of pulse periods in the north–south direction from the 1994 Northridge earthquake of (a) CSM simulations and (b) EXSIM simulations compared with recordings.



Figure 25. Empirical cumulative distribution function of structural collapse capacity using PEER records and matched simulations on nonlinear SDOF structures with (a) T = 0.3 s, (b) T = 0.8 s, and (c) T = 2 s. The color version of this figure is available only in the electronic edition.

correlations matched empirical models at long periods for which simulations are deterministically computed but were underestimated at short periods for which simulations are stochastically generated. The ratio of $SA_{RotD100}$ to SA_{RotD50} is a proxy for the polarization of ground motions and is important for 3D structures. In our examples, the simulations tend to be more polarized than recordings at long periods, but the simulation methods give varying results at short periods. The inelastic-to-elastic displacement ratio is a proxy for the response of nonlinear structures, and most structures are designed to behave nonlinearly in a large earthquake. This ratio is highly dependent on the mean and standard deviation of the elastic response spectra at longer periods because the structure's period effectively lengthens as the structure behaves nonlinearly, and we observed that this ratio from recordings and simulations is similar when the mean and standard deviation match.

Then, we discussed other parameters such as directivity features and collapse capacity of idealized structures because they are important for engineering applications, although they are not as well constrained by empirical models and are thus more difficult to validate. Because relatively few recordings exist that contain directivity pulses, empirical models may not be as reliable as simulations in predicting pulse periods. However, in the example simulations, we observed that directivity parameters such as presence of a pulse and pulse period vary significantly between simulation methods, making it difficult to know which is physically correct. Collapse capacity is important for the evaluation of highly nonlinear structures and for seismic risk assessments, but it is highly dependent on ground-motion selection and structural modeling choices. Therefore, in validating this parameter, great care must be taken to choose ground-motion recordings and simulations with similar elastic spectra, and the same structural model must be evaluated for all groundmotion sets. To demonstrate the validation framework, we computed the collapse capacity of idealized structural models and observed that the median collapse capacity from simulations was within 12% of recordings.

In general, we observed that each simulation method matched some empirical models and not others, indicating the value of checking the accuracy of all relevant groundmotion parameters. The list of ground-motion parameters proposed in this study is not exhaustive but covers some proxies that are important to the seismic response of buildings for a variety of reasons. For other engineering applications (e.g., landslide analysis, distributed systems, etc.), the use of other relevant proxies is important; and, for applications where no relevant proxies exist (e.g., building damage due to basin effects or fling step), a different validation strategy is necessary. Further research may be necessary to quantify how similar simulations should be to an empirical model to ensure reliable results, but in many cases, the validation framework presented here is consistent and can provide confidence in simulations for appropriate applications.

Data and Resources

Simulations were either obtained from the Southern California Earthquake Center (SCEC) Broadband Platform (BBP) validation exercise (Dreger *et al.*, 2014) or computed using SCEC BBP version 11.2.3, which is available for download at http://scec.usc.edu/scecpedia/Broadband_Platform (last accessed July 2013). Recordings were obtained from the Pacific Earthquake Engineering Research Center Next Generation Attenuation database at http://peer.berkeley.edu/ peer_ground_motion_database (last accessed July 2013).

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