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# INTEGRATION OF SITE-SPECIFIC GROUND RESPONSE ANALYSIS RESULTS INTO PROBABILISTIC SEISMIC HAZARD ANALYSES

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## ABSTRACT

Site specific ground response analyses have the potential to improve the accuracy of ground motion hazard estimates. We implement such analyses into probabilistic seismic hazard analyses (PSHA), which overcomes limitations of previous practice in which site-specific amplification factors were often applied deterministically. The implementation involves running nonlinear ground response analyses using multiple input rock motions spanning a wide range of source/path characteristics and amplitudes through a site profile and calculating amplification factors (*AF*) for each motion at periods of interest. A specific functional form is regressed through these data points and the standard deviation of the fit is established. The median amplification level is coupled with the median rock motion (from ground motion prediction equations) to estimate median soil motions. Two alternative estimates of standard deviation are possible - one is empirically based and the other is theoretically derived. The modified median and standard deviation are then used inside of the hazard integral. The impact on PSHA results of site-specific versus generic site-factors is shown to be large for a soft soil site for *PGA* and 1.0 sec spectral acceleration and modest for a stiff soil site. A significant fraction of the difference between PSHA results for different site models results from factors other than the differences in median amplification factors, highlighting the importance of formal probabilistic analysis of site effects in lieu of applying ad hoc multipliers to results for reference site conditions.

Keywords: ground response analysis, site effects, seismic analysis, OpenSHA.

## INTRODUCTION

Probabilistic seismic hazard analyses (PSHA) are performed using empirical ground motion prediction equations (GMPEs) in combination with earthquake source models. The GMPEs provide estimates of the median and standard deviation of a ground motion intensity measure (IM) conditioned on various source, path, and site parameters. Geotechnical ground response analyses, generally using one-dimensional (1-D) modeling, have the potential to improve ground motion estimates relative to those provided by GMPEs, because they consider site-specific attributes of the site response. In this paper, we discuss the problem of linking ground response analysis results with PSHA.

Past practice on this issue has been deterministic. For example, some have applied IM amplification factors derived from ground response analysis to ground motion levels evaluated for a reference site condition (typically rock) from PSHA. Unfortunately, this practice neglects the uncertainty in the site response and the effect of site response on the uncertainty of IM. Lacking

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formal consideration of these factors, the deterministic application of amplification factors results in a surface ground motion prediction with an unknown rate of occurrence.

In this article we describe tools that have been implemented within OpenSHA (http://www.opensha.org/) that allow ground response analysis results to be formally integrated into PSHA. Through two examples we provide guidance on the proper use of the tools, the effects of using site-specific analyses in lieu of generic site factors, and the limitations of the present implementation.

## METHODS FOR IMPLEMENTING GROUND RESONSE RESULTS IN PSHA

Two general approaches are available for implementing ground response analysis results in PSHA. The first one (Bazzurro and Cornell, 2004) involves convolution of frequency-dependent amplification factors (*AF*) calculated from ground response analysis with a site hazard curve developed for reference site conditions (usually rock). The second procedure involves modifying the median and standard deviation of a ground motion intensity measure estimated from a rock attenuation relationship (Baturay and Stewart, 2003; Bazzurro and Cornell, 2004). For reasons provided subsequently, we focus in this article on the second procedure.

For a given earthquake scenario the median and standard deviation of the spectral acceleration on rock are taken from a GMPE and are denoted $\hat{S}_a^r(f)$ for the median and $\sigma_{\ln S_a^r(f)}$ for the standard deviation. Ground response analyses are performed using an appropriate equivalent-linear or nonlinear code using multiple rock input motions (possibly scaled to different levels) and (if desired) multiple versions of the site soil profile to account for variability in soil properties. Although some 1D ground response analysis computer codes allow the use of more than one component of the same recording, this methodology was originally developed using one randomly selected horizontal component. The results consist of a suite of calculated ground surface motions. The ratio of the spectral accelerations of a ground surface motion to its corresponding rock motion is taken as an amplification factor (*AF),* which is evaluated for frequency *f* as,

$$AF(f) = \frac{S_a^s(f)}{S_a^r(f)} \tag{1}$$

where $S_a^r(f)$ is the (input) rock spectral acceleration at frequency *f* and $S_a^s(f)$ is the (output) soil spectral acceleration. An example of the relationship between $S_a^r(f)$ and *AF(f)* is shown in Figure 1, where each cross represents the results of a ground response analysis for a rock record. The decrease of *AF(f)* with $S_a^r(f)$ is due to sediment nonlinearity. It is often assumed that in order to estimate the expected *AF(f)* at a given frequency *f*, one needs to know only the value of $S_a^r(f)$, as has been found for certain aspects of nonlinear structural response (Shome et al., 1998). If this assumption is adopted, no other information about the spectrum of the input motion and about the characteristics of the causative event (e.g., the magnitude) is strictly necessary.

A non-linear fit curve can be established through the data, as illustrated in Figure 1. An equation that has proved to be broadly applicable is:

$$\ln AF(f) \approx c_0 + c_1 \ln\left(S_a^r(f) + c_2\right) + \varepsilon_{\ln AF^{Sa}(f)} \sigma_{\ln AF^{Sa}(f)} \tag{2}$$

where $c_0$, $c_1$, and $c_2$ are regression coefficients; $\sigma_{\ln AF(f)}$ is the standard deviation of the *AF(f)* conditional on $S_a^r(f)$ (i.e., the standard error of estimation from the statistical regression); and

$\varepsilon_{\ln AF^{Sa}(f)}$ is a standard normal variate. Note that, for readability, we dropped *r* as a reference to rock conditions from $S_a(f)$ when it appears as superscript above in the $\sigma_{\ln AF^{Sa}(f)}$ and $\varepsilon_{\ln AF^{Sa}(f)}$ terms.

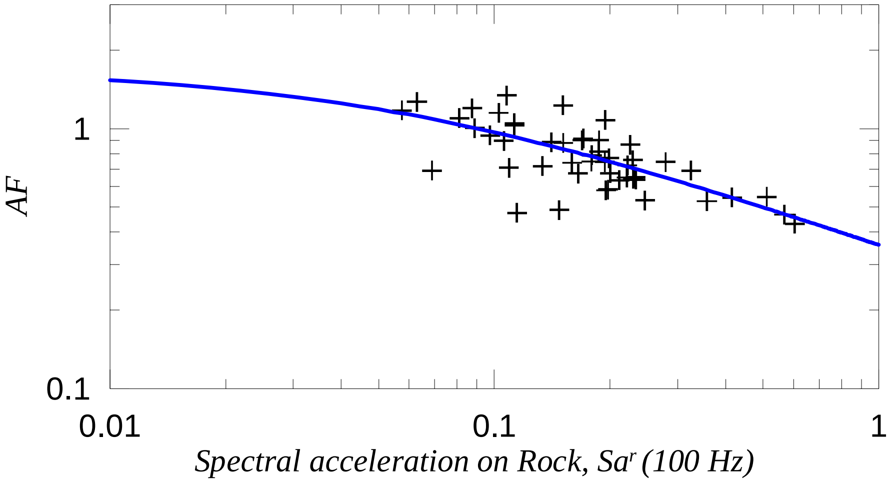


**Figure 1. Example of individual site amplification factors and fit curve calculated from ground response analyses plotted against reference motion amplitude.**

Once the regression represented by Eq. 2 has been completed, the median of the soil motion ($\hat{S}_a^s(f)$) for a given earthquake scenario can be evaluated from the median *AF* and the median $S_a^r$ as:

$$\ln \hat{S}_a^s(f) \approx \ln(\hat{S}_a^r) + c_0 + c_1 \ln(\hat{S}_a^r(f) + c_2) \quad (3)$$

Note that the last two terms correspond to the median *AF(f)* from Equation 2.

Two approaches can be used to estimate the standard deviation of the spectral acceleration on soil ($\sigma_{\ln S_a^s(f)}$). The first approach involves the separation of standard deviation into intra-event and inter-event terms (consistent with mixed-effects regression; Abrahamson and Youngs, 1992). The intra-event standard deviation (generally termed $\sigma$) is derived from the residuals obtained when a "model" is applied to "data". The model in this case is a hybrid of a GMPE for rock coupled with ground response analyses and an event term to remove event-specific bias.

The "data" consists of recordings to which the above model is applied. The data set is much smaller than what is typically used to develop ground motion prediction equations (GMPEs) because detailed geotechnical profiles are required for each site in the database. Baturay and Stewart (2003) performed such analyses and expressed the intra-event variance as the sum of the variance of residuals (termed $\sigma_{g-net}^2$) and the square of the standard error of the *AF*s ($se^2 = \sigma_{AF(f)}^2 / n$, where *n* is the number of runs of ground response analyses for a given site). Quantity $\sigma_{g-net}^2$ was found to vary with site condition as shown in Figure 2. This intra-event dispersion is then combined with inter-event standard deviation ($\tau$) to evaluate the total error for use in PSHA:

$$\sigma_{\ln S_a^s(f)} = \sqrt{\sigma_{g-net}^2 + Se^2 + \tau^2} \quad (4)$$

Note that this error is not conditional on $S_a^r$

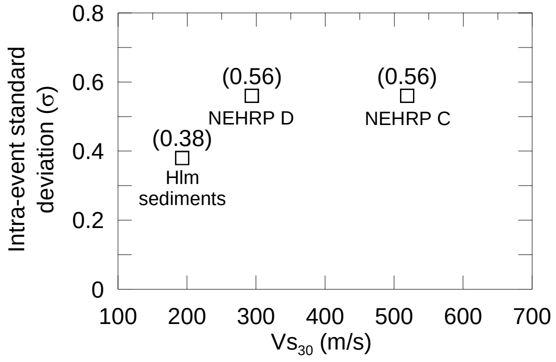


**Figure 2. Standard deviation of spectral acceleration at the soil surface at high frequency (3 Hz) associated with a ground motion prediction model that includes ground response analysis (Baturay and Stewart, 2003). Hlm indicates sediments of Holocene age and lacustrine or marine origin.**

The philosophy behind the second approach, originally presented by Bazzurro and Cornell (2004), is to derive a standard deviation term based on the application of the nonlinear operator for the site (*AF(f)*) to the lognormally distributed rock motion. Bazzurro and Cornell (2004) showed that for the case of linear *AF(f)* function (i.e., $c_2$=0) that the standard deviation on soil can be taken as:

$$\sigma_{\ln S_a^s(f)} = \sqrt{(c_1+1)^2 \sigma^2_{\ln S_a^r(f)} + \sigma^2_{\ln AF^{Sa}(f)}} \tag{5}$$

It is interesting to note that $c_1$ is often negative (as is the case in Fig. 1), which can cause $\sigma_{\ln S_a^s(f)}$ to be less than or equal to $\sigma_{\ln S_a^r(f)}$. The use of a non-linear functional form for *AF(f)* requires replacing in Eq. 5 the $c_1$ term by an $S_a^r$-dependent slope term that accounts for the curvature showed on Fig. 1. The implementation of this feature in OpenSHA is currently in progress. For the implementations discussed in this paper, we have used Eq. 5 as an approximation of the correct standard deviation term. Note that for the larger $S_a^r$ values typically of interest in PSHA for design purposes, Eq. 5 provide a good approximation. It is important to note that in this approach, as currently implemented, $\sigma_{\ln Sa^r}$ includes both inter- and intra-event variability. Hence, both sources of dispersion are reduced by the site nonlinearity through the $c_1$ term in Eq. 5. Further discussion of this issue is provided in the conclusions.

The modified median and standard deviation obtained from Eqs. 3-5 can be used directly within the hazard integral in lieu of those from a conventional GMPE. The use is legitimate provided that the definition of $S_a^r(f)$ used in the rock GMPE and for the prediction of *AF(f)* is consistent. As originally discussed in Baker and Cornell (2006) for structural response studies, a mismatch can occur, for example, if the attenuation relationship is developed for the geometric mean of $S_a^r(f)$ for the two horizontal components and the parameter used as the predictor of *AF(f)* is instead the $S_a^r(f)$ of an arbitrary component used for ground response analysis. The mismatch occurs because the dispersion of a ground motion parameter's geometric mean is smaller than the dispersion of the same parameter from an arbitrary component. It is therefore important to be consistent with the type of ground motion (geometric mean or arbitrary component) utilized for PSHA and for ground response analysis. At the present time, the dispersion has not been modified for these effects.

## OVERVIEW OF OPENSHA

OpenSHA is an open-source object-oriented toolbox developed to provide the engineering community with state of the art tools to conduct seismic hazard analyses (Field et al., 2005). OpenSHA is web-based and freely available at www.opensha.org where Java applications can be downloaded. For this paper, we will focus on the Hazard Curve and the Attenuation Relationship applications (the former is shown in Figure 3) in which the above approaches have been implemented.

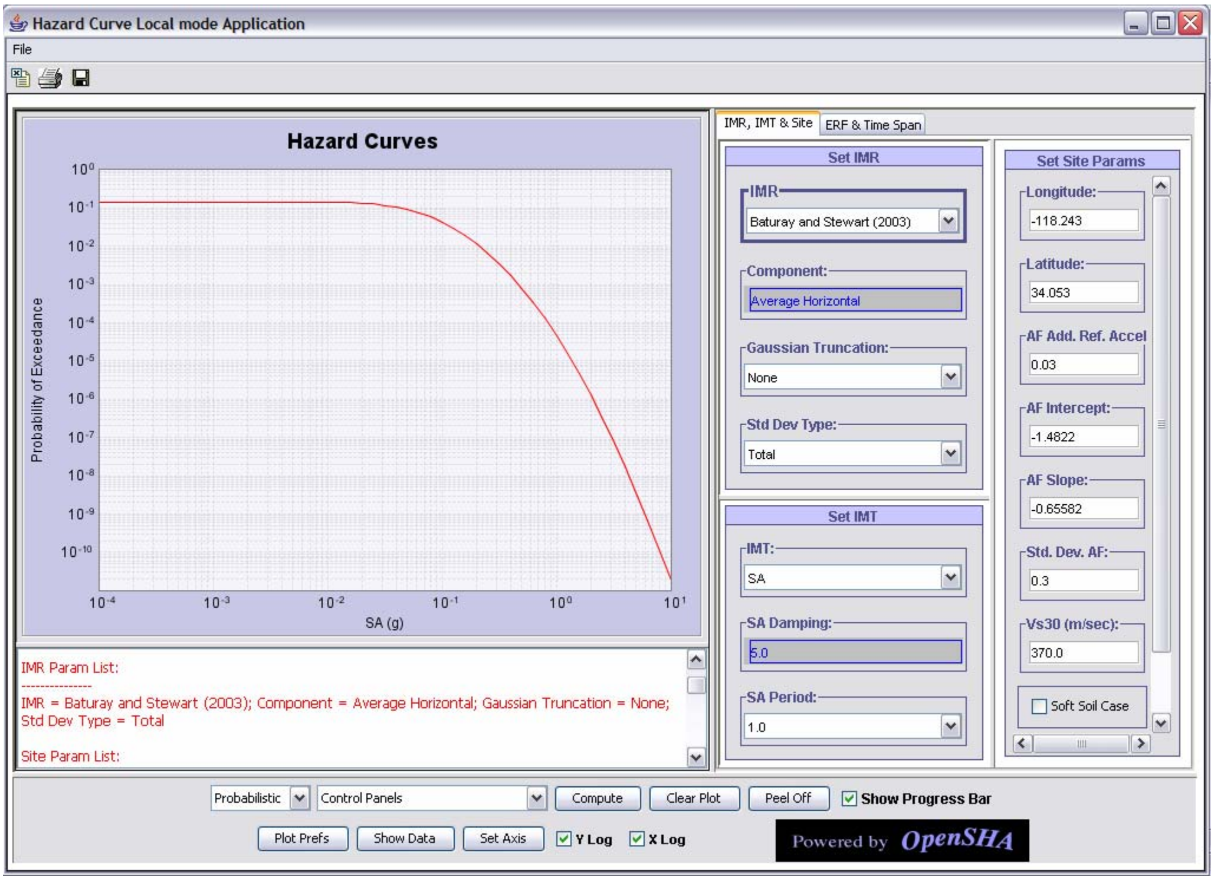


**Figure 3. Screen shot of the OpenSHA Hazard Curve application. The Graphic User Interface (GUI) allows the user to quickly access and modify the variables for each analysis.**

In the current implementation, the user inputs the *AF(f)* parameters introduced in Eq. 2: $c_0$, $c_1$, $c_2$ (often taken as 0.03g), and $\sigma_{\ln AF(f)}$. Each of these terms are derived from the regression of site-specific ground response analyses results (e.g., as illustrated in Fig 1). Additional parameters for Baturay and Stewart (2003) are the number of runs used in the ground response analyses *n* and the average shear wave velocity for the upper 30 meters of soil column, $V_{s30}$. The "soft soil" option can also be used to override $V_{s30}$ to specify the "Hlm" condition (see caption of Fig. 2) for analysis of $\sigma_{g-net}$.

One implementation in OpenSHA is referred to as "Bazzurro and Cornell (2004)" and uses Eq. 3 and 5 to define the median and standard deviation used inside the hazard integral. The second OpenSHA implemention is termed "Baturay and Stewart (2003)" and uses Eq. 3 and 4, with the following specifications:

- For T<0.75 seconds: use Eq. 3 and 4 as presented above

- For T>1.5 seconds: use the median and standard deviation from Choi and Stewart (2003). This is done because the empirical data do not support the use of 1-D analysis at long periods.
- For 0.75<T<1.5: interpolate a linear fit between the two ending points

The computations outlined in the bullets above are performed internally within the Hazard Curve application.

## ILLUSTRATION THROUGH EXAMPLES

We now illustrate the application of OpenSHA with site-specific ground response analyses for the two example site profiles listed in Table 1. Within the hazard application, one can select to run either a single scenario (one specific fault segment rupture) or use the regional predefined fault scenarios (e.g. Frankel 2002). Other options include the earthquake source model and time span ($\Delta t$) over which the probability of exceedance of the surface ground motion is computed. Here we used probabilistic analysis with the Frankel 2002 source model with $\Delta t$=1.0 year. Background seismicity is excluded, and the site's geodetic coordinates are taken as Lat 34.053 N, Long -118.243 W (default in OpenSHA). Both site profiles are utilized at this location.

**Table 1. Sites profiles used for ground response analysis**

| Original location and site name | $V_{s30}$ (m/s)[1] | Geology |
|---|---|---|
| Emeryville, Pacific Park Plaza | 198 | Marine Clay |
| Los Angeles, Sepulveda VA | 370 | Deep alluvium |

[1] Average shear wave velocity in upper 30 m of site

A uniform set of 49 outcropping acceleration records were used as input, each of which was recorded on soft rock site conditions in California (NEHRP B and C site categories). Characteristics of the input motion suite are shown in Figure 4. The ground response analyses were performed using the code SUMDES (Li et al., 1992) with the total stress analysis module (SUMDES Model 6). Because PSHA samples over a wide range of magnitudes and distances, the input motions utilized in ground response analyses should sample over a similarly broad range, with a corresponding large variability in $PGA^r$ values. For example, there are weak motions from small earthquakes (which can control for high annual probabilities) and strong motions from larger earthquakes (which can correspond to lower probabilities). One aspect not considered during the selection of input motions is $\varepsilon$ (offset between data and GMPE median normalized by GMPE standard deviation), the effects of which on ground response analysis results are currently unknown.

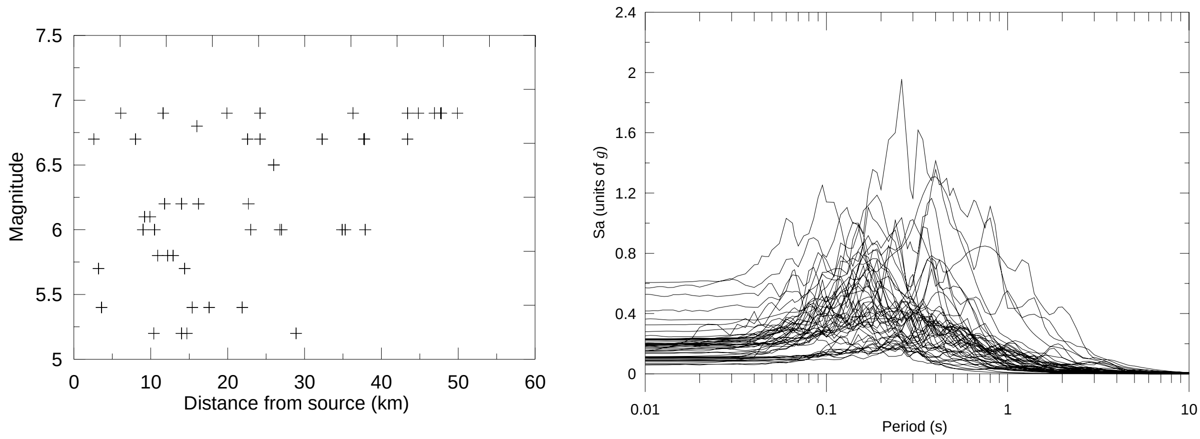


**Figure 4. Properties of selected ground motions used as input for SUMDES analyses. a) Records plotted in magnitude-distance space. b) Spectra for all 49 selected motions.**

Small strain soil shear moduli ($G_{max}$) were computed from in situ measurements of shear wave velocity. Other backbone curve parameters and viscous damping parameters were selected based on the nonlinear code implementation guidelines given in Stewart et al. (2006).

The results of PSHA for the two site profiles are presented in Figure 5. For each site, hazard curves are shown for the geometric mean of the horizontal peak acceleration and 1.0 sec elastic spectral acceleration (for 5% damping). Hazard curves are shown for the following GMPEs for each site:

1. Rock site conditions (using the rock GMPE from Abrahamson and Silva, 1997);
2. Generic soil (using the soil GMPE from Abrahamson and Silva, 1997);
3. Generic sites with $V_{s30}$ matching the values at the subject sites (using GMPE by Choi and Stewart, 2005).
4. Site specific (using the criteria described in this article, coupled with the rock GMPE by Abrahamson and Silva, 1997).

Referring initially to the *PGA* results for Emeryville, the rock motions (red curve) provide the lowest ground motions at low return periods (i.e., low annual probability of exceedance, APE) but the highest at long return periods. This difference is due to the nonlinearity inherent to each of the site amplification models considered. At the low APE levels typically used in engineering design (2% or 10% probability of exceedance in 50 years, see Figure 5 legend), the site-specific analysis provides the lowest *PGA* estimates, followed by the $V_{s30}$-based nonlinear site amplification model (Choi and Stewart, 2005) and the generic site amplification model of Abrahamson and Silva (1997).

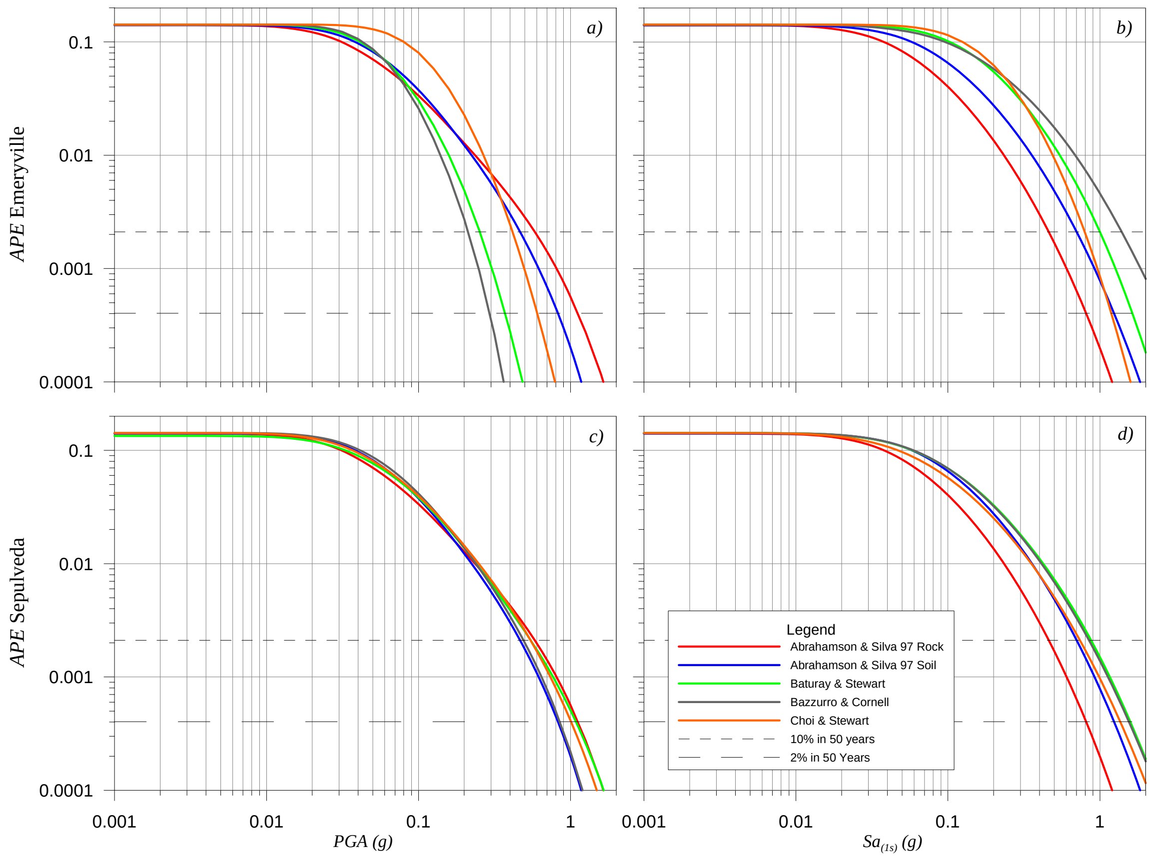


**Figure 5. Hazard curves for Emeryville (*a* and *b*) and Sepulveda (*c* and *d*) site profiles for *PGA* and 1.0 sec $S_a$.**

The *PGA* results for Sepulveda are much more similar to each other, because the site profile at Sepulveda is much nearer the “average” condition at soil sites in empirical ground motion databases. This causes the $V_{s30}$-based model and the site specific analyses to produce results similar to the generic site term of Abrahamson and Silva (1997). For 1.0 sec $S_a$, the site amplification is relatively linear, and the rock hazard curve is the lowest of all models across all hazard levels shown. We note also that there is significantly less 1.0 sec site amplification at Sepulveda (Figure 5*d*) than for the relatively soft Emeryville site (Figure 5*b*).

To explore the differences between prediction models in more depth, we re-plot in Figure 6 the site hazard curves for *PGA* at Emeryville along with the median site amplification factors (relative to soft rock) for each of the three models considered (generic soil, $V_{s30}$-based amplification and site-specific analysis). The amplification factors are plotted as a function of the corresponding rock *PGA* ($PGA^r$). The ratio of the soil (site-specific) to rock motions at an APE of 10% in 50 years is approximately 0.43 (0.25g/0.58g). For the 0.58 g rock ground motion level, the corresponding median *AF* is 0.33 (Figure 6*b*). Similarly, the ratio of soil ($V_{s30}$-based) to rock motions at this hazard level is approximately 0.7 (0.41g/0.58g), which can be compared to a median amplification level of 0.45. Hence, a good deal of the reduction in *PGA* for the soil-specific case can be explained by the lower median of *AF*. The remaining difference, which is discussed in the following paragraphs, results from different source characteristics critically affecting the hazard for the two GMPE models coupled with different standard deviation terms.

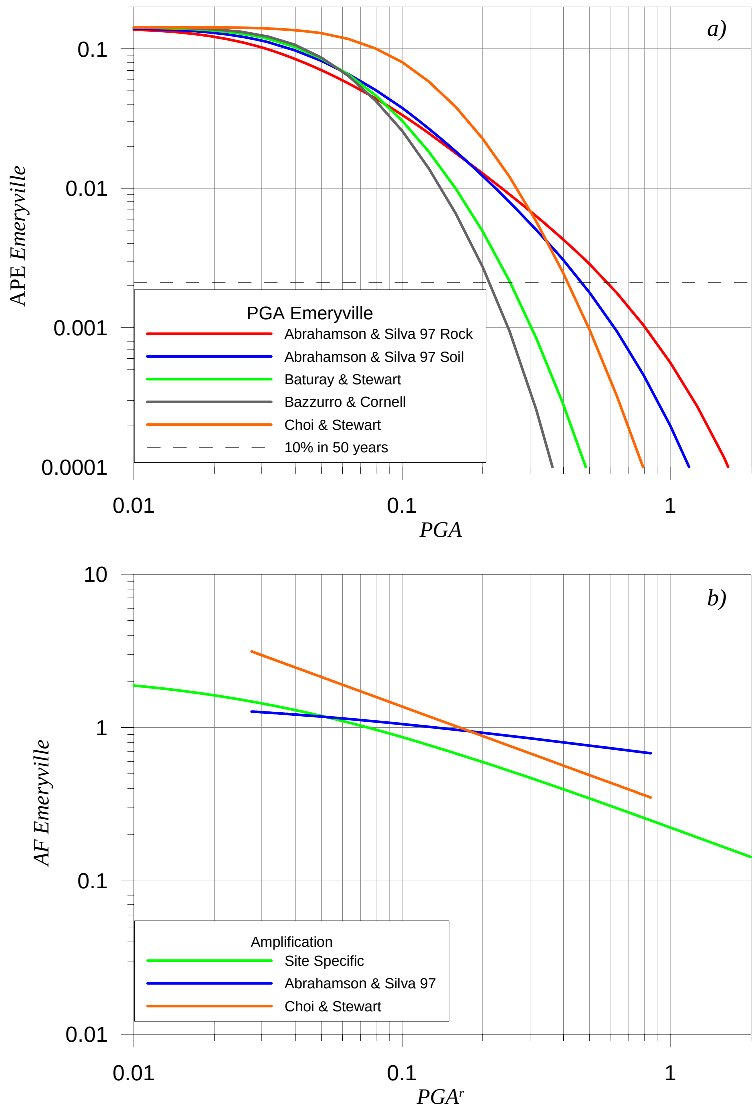


**Figure 6. *PGA* hazard curves and corresponding median amplification functions for Emeryville.**

We have investigated the source of the differences observed above. For a given magnitude and distance, the GMPE models produce difference medians and standard deviations, but for a given hazard level, the characteristic of the contributing sources are also different. To assess these differences, we use the OpenSHA disaggregation tool for the 10% in 50 years hazard level (equivalent APE of 0.0021) for each GMPE model. Figure 7 shows the results of the disaggregation, and also shows the weighted (by the relative contributions) average of the magnitude $\overline{M}$ , distance $\bar{r}$ (in km) and epsilon $\bar{\varepsilon}$ characteristics.

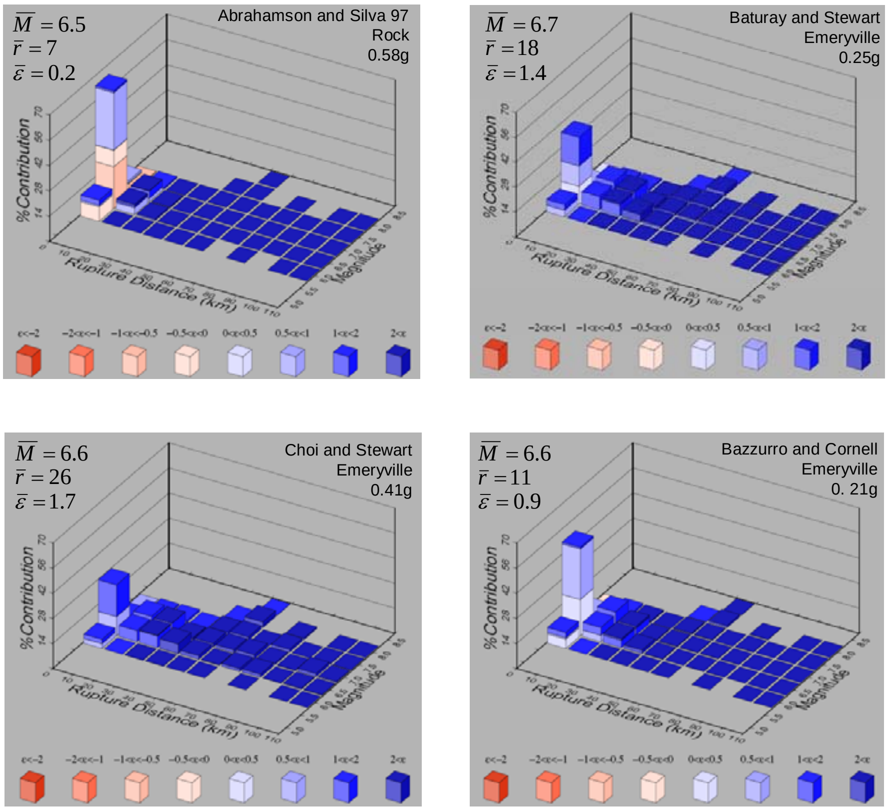


**Figure 7. Comparison of hazard disaggregation results for the 10% in 50 years *PGA* hazard, Emeryville site conditions.**

The *PGA* disaggregation results for the Abrahamson & Silva rock case show a fairly narrow range of controlling magnitudes and distances (i.e., a large percentage of the overall hazard comes from $M$ = 6.0-6.5 and $r$ = 0-10 km) while the epsilon values are generally less than 1.0. On the other hand for soft soils as represented by the Choi & Stewart GMPE, the controlling sources are more broadly distributed, especially in terms of distance (i.e., significant contributions occur to distances out to 40 km). This is due to large soil amplification of relatively weak rock input motions, such as those generated by earthquakes at large distances. Moreover, the epsilon values are higher than for the rock case, suggesting that more rare realizations of ground motions (well above the median) control the hazard on soft soil.

The question remains, what factors control the different ground motion estimates for a given hazard level for rock versus soft soil. There are three principal possibilities: (1) different median motions (resulting from the amplification factor for soil); (2) different standard deviation terms for rock and soil; and (3) different controlling sources, as noted above. We illustrate the impact of these factors with a numerical example. Using the OpenSHA Attenuation Relationship applet, we obtain a median $\mu$ and standard deviation $\sigma$ of *PGA* both for rock and soil for equivalent single-source scenarios defined from disaggregation ($\overline{M}$ , $\bar{r}$ ), as shown in Table 2. We then compute a deterministic *PGA* value combining the median, the standard deviation and the mean epsilon value as follows:

$$PGA = \mu e^{\bar{\varepsilon}\sigma} \quad (6)$$

Results of this computation are shown in Table 2. Despite the crudeness of the approximation of a hazard curve ordinate by these single-source scenarios, the deterministic *PGA* values are within 15 % of the probabilistic *PGA* values.

**Table 2. Deterministic computation of *PGA* at the 10% in 50 years hazard level**

| GMPE | *PGA*[1] (PSHA) | $\mu = F(\overline{M}, \bar{r})$ | $\sigma = F(\overline{M}, \bar{r})$ | $\bar{\varepsilon}$ | *PGA*[2] |
|---|---|---|---|---|---|
| Abrahamson & Silva 1997 Rock | 0.58 | 0.43 | 0.50 | 0.2 | 0.50 |
| Choi & Stewart, Emeryville | 0.41 | 0.15 | 0.52 | 1.7 | 0.41 |
| Baturay & Stewart, Emeryville | 0.25 | 0.12 | 0.55 | 1.4 | 0.26 |
| Bazzurro & Cornell, Emeryville | 0.21 | 0.14 | 0.34 | 0.9 | 0.19 |

[1] Values reported from Figure 7, [2] Values computed with Equation 6

Examining the results in Table 2, a significant contributor to the difference in the ground motion estimates from the Abrahamson & Silva rock and Choi & Stewart soil GMPEs is the different medians. The differences in the medians results from different average distances (7 km for rock vs. 26 km for soft soil, as shown in Fig. 7), which produces partially offsetting effects of reduced motions for reference rock and increased site factors (increasing the soil median). The net effect of these two factors is a reduced soil median relative to rock, as shown in Table 2. Comparing Abrahamson & Silva to Choi & Stewart, we find similar standard deviations, mostly because Choi & Stewart has a much larger inter-event variability term compensated by a smaller intra-event variability term (not shown here). This is a by-product of the different ages of the models. The median from Choi & Stewart is approximately one-third of the median from Abrahamson & Silva, yet the ratio of the ground motions from the hazard analysis is 0.7 (Figure 7). The difference in these ratios (1/3 for the median vs 0.7 for probabilistic result) comes from the different $\bar{\varepsilon}$ values, which are much larger for soft soil than rock. This tends to increase the *PGA* for soft soil, bringing it closer to the value for rock. In summary, the differences in the rock and soft soil hazard estimates results from the nonlinear site factor for soft soil, larger contributions from relatively distant sources for soft soil, and relatively large $\bar{\varepsilon}$ values for soft soil.

The Choi & Stewart (generic site factor) and Baturay & Stewart (site-specific site factor) can similarly be compared. We observe that the differences in all the components are much smaller. Whereas the median for Choi & Stewart is slightly larger than the one from Baturay & Stewart, the product of the standard deviations with their respective values of $\bar{\varepsilon}$ is almost identical. In this case, it is mainly the difference in median values (driven by different distances) that can explain the observed differences in the *PGA*s from the hazard analysis.

A similar comparison can be made for the site-specific models of Baturay & Stewart and Bazzurro & Cornell. The most important contributor to the hazard differences between these two is their different standard deviations, which ultimately drives down the *PGA* value for Bazzurro & Cornell relative to Baturay & Stewart.

## CONCLUSIONS

Past practice in seismic hazard evaluation has generally been that site-specific amplification functions are not considered in hazard estimation or they are incorporated in an ad-hoc manner that fails to preserve the desired probability level (i.e., return period) of the surface motions. The implementation of modules for incorporating the results of ground response analysis into OpenSHA provides the opportunity to significantly advance seismic hazard assessment in this regard. Preliminary results suggest that these effects are most pronounced for soft soils, where site effects can exhibit critical site-specific attributes that are not well captured by generic site factors. We have illustrated the differences in hazard results for two generic and two site-specific GMPEs. We have observed that the differences in the models (median and standard deviation) have an impact on the ground motion hazard. The disaggregation tool implemented within OpenSHA can be very useful in the interpretation of hazard results that might appear at first to be counter-intuitive.

The current implementation in OpenSHA is preliminary and work is on-going to improve it. There are two significant technical issues that remain under investigation. One concerns the dependence of the standard deviation term of the *AF* given $S_a^r$ on the slope of the *AF*-reference motion amplitude function. The current implementation is based on the constant slope (linear model) at large input motion levels which underestimates the dispersion at lower levels. This can be explained by looking at Eq. 5 where the slope ($c_1+1$) term multiplies the standard deviation $\sigma_{\ln Sa^r}$ term. Using the linear assumption for the slope, which is negative, there is an effective reduction in dispersion. If the slope were reduced for smaller input motion levels, the dispersion would be increased. It is this dependence of dispersion on input motion amplitude that is not currently considered. An alternative functional form to Eq. 5 that considers the true slope of the curve is under development to account for this effect. The second issue concerns the treatment of inter-event variability in the "second approach" for dispersion estimation adapted from the original work of Bazzurro and Cornell (2004). At present, inter-event variability is reduced by the slope term as indicated in Eq. 5; this may be modified in the future so that only intra-event variability is reduced in the manner suggested by Eq. 5. Other relatively minor improvements we expect to make relate to implementing multiple options of rock GMPEs that could be used with the site-specific amplification models and adding an option that would allow a more flexible transitioning from site-specific to empirical site factors as period increases.

As a final comment, it is noted that the SUMDES simulations presented here may underestimate *AF* for high-frequency ground motion parameters such as *PGA* due to the formulation of the viscous damping model. Future work will utilize an alternative nonlinear site response analysis code that does not have this limitation.

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