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DEVELOPMENT OF REGIONAL HARD ROCK ATTENUATION RELATIONS FOR CENTRAL AND EASTERN NORTH AMERICA

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Background

Due to the low rates of seismicity, a significant and currently unresolvable issue exists in the estimation of strong ground motions for specified magnitude, distance, and site conditions in central and eastern North America (CENA). The preferred approach to estimating design ground motions is through the use of empirical attenuation relations, perhaps augmented with a model based relation to capture regional influences. For western North America (WNA), particularly California, seismicity rates are such that sufficient strong motion recordings are available for ranges in magnitudes and distances to properly constrain regression analyses. Naturally, not enough recorded data are available at close distances ($\leq$ 10 km) to large magnitude earthquakes (**M** $\geq$ 6 3/4) so large uncertainty exists for these design conditions but, in general, ground motions are reasonably well defined. For CENA however, very few data exist and nearly all are for **M** $\leq$ 5.8 and distances exceeding about 50 km. This is a fortunate circumstance in terms of hazard but, because the potential exists for large, though infrequent, earthquakes in certain areas of CENA, the actual risk to life and structures is comparable to that which exists in seismically active WNA. As a result, the need to characterize strong ground motions is significant and considerable effort has been directed to developing appropriate attenuation relations for CENA conditions (Boore and Atkinson, 1987; Toro and McGuire, 1987; EPRI, 1993; Toro et al., 1997; Atkinson and Boore, 1997). Because the strong motion data set is sparse in the CENA, numerical simulations represent the only available approach and the stochastic point-source model (Appendix A) has generally been the preferred model used to develop attenuation relations. The process involves repeatedly exercising the model for a range in magnitude and distances as well as expected parameter values, adopting a functional form for a regression equation, and finally performing regression analyses to determine coefficients for median predictions as well as variability about the median. Essential elements in this process include: a physically realistic, reasonably robust and well-validated model (Silva et al., 1997; Schneider et al., 1993); appropriate parameter values and their distributions; and a statistically stable estimate of model variability (Appendix A). The model variability is added to the variability resulting from the regression analyses (parametric plus regression variability) to represent the total variability associated with median estimates of ground motions (Appendix A).

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Model Parameters

For the point-source model implemented here, parameters include stress drop ($\Delta\sigma$), source depth (H), path damping ($Q(f) = Q_o f^{\eta}$), shallow crustal damping (kappa), and crustal amplification. For the regional crust, the Midcontinent model from EPRI (1993; also in Toro et al., 1997) was adopted. The crustal model is listed in Table 1. The Moho is at a depth of about 40 km. Geometrical attenuation is assumed to be magnitude dependent, using a model based on inversions of the Abrahamson and Silva (1997) empirical attenuation relation with the point-source model. The model for geometrical attenuation is given by

$$R^{-(a + b(\mathbf{M} - 6.5))}, \; R \leq 80 \text{ km}; \quad R^{-(a + b(\mathbf{M} - 6.5))/2}, \quad R > 80 \text{ km} \qquad (1)$$

where a = 1.0296, b = -0.0422, and 80 km reflects about twice the crustal thickness (Table 1).

The duration model is taken as the inverse corner frequency plus a smooth distance term of 0.05 times the hypocentral distance (Herrmann, 1985). Monotonic trends in both the geometrical attenuation and distance duration models produced no biases in the validation exercises using WNA and CENA recordings (Appendix A) and are considered appropriate when considerable variability in crustal structure that may exist over a region, as well as variability in source depth. Additionally, extensive modeling exercises have shown that the effects of source finiteness, coupled with variability in source depth and crustal structure, result in smooth attenuation with distance, accompanied by a large variability in ground motions (EPRI, 1993).

To model shallow crustal damping, a kappa value of 0.006 sec is assumed to apply for the crystalline basement and below (Silva and Darragh, 1995; EPRI, 1993). The Q(f) model is from Silva et al. (1997), based on inversions of CEUS recordings and is given by $Q(f) = 351 f^{0.84}$. Both magnitude independent and magnitude dependent stress drop models are used. For the magnitude dependent stress drop model, the stress drop varies from 160 bars for **M** 5.5 to 90 bars for **M** 7.5 and 70 bars for **M** 8.5 (the range in magnitudes for the simulations). The magnitude scaling of stress drop is based on point-source inversions of the Abrahamson and Silva (1997) empirical attenuation relation (Silva et al., 1997) and is an empirically driven mechanism to accommodate the observed magnitude saturation due to source finiteness. Similar point-source stress drop scaling has been observed by Atkinson and Silva (1997) using (WNA) recordings of strong ground motions and from inversions of the Sadigh et al., (1997) attenuation relation (EPRI, 1993). For the CEUS, the stress drop values are constrained by the **M** 5.5 stress drop of 160 bars. This value is from recent work of Gail Atkinson (personal communication, 1998) who determined CENA stress drops based on instrumental and intensity data. Since the majority of her data are from earthquakes below **M** 6 (**M** 4 to 7), it was assumed her average stress drop ($\approx$ 180 bars adjusted for the regional crustal model to 160 bars) is appropriate for **M** 5.5. Table 2 shows the magnitude dependent as well as magnitude independent stress drops. The magnitude independent stress drop of 120 bars reflects the log average of the **M** 5.5, **M** 6.5, and **M** 7.5 stress drops (Table 2).

Source depth is also assumed to be magnitude dependent and is based on the depth distribution of stable continental interiors and margins (EPRI, 1993). The magnitude dependent depth distribution is shown in Table 2.

The single corner frequency model was also run with a constant stress drop for all magnitudes. A stress drop of 120 bars was applied to all four magnitudes. This is the same constant stress drop used in the Toro et al. (1997; EPRI, 1993) CEUS hard rock relation.

Another source model considered acceptable for CENA ground motions is the double corner model (Atkinson and Boore, 1995). In this model there is no variation of the stress drop with magnitude. Additionally, stress drop is not explicitly defined for this model and no uncertainties are given for the corner frequencies (which are magnitude dependent). As a result, the parametric uncertainty obtained from the regression analysis will underepresent the total parametric uncertainty. For this reason, the total parametric uncertainty for the two-corner model is taken as the total parametric uncertainty from the single corner model with variable stress drop, which is slightly larger than the parametric uncertainty for the single corner model with constant stress drop scaling (to avoid underestimating the two-corner parametric uncertainty).

To accommodate magnitude saturation in the double-corner and single-corner constant stress drop models, magnitude dependent fictitious depth terms were added to the source depth. The functional form is given by

$$H = H' \, e,^{a + bM} \qquad (2)$$

with

$$a = -1.250, \qquad b = 0.227.$$

H and H' are the fictitious and original source depths respectively and the coefficients are based on the Abrahamson and Silva (1997) empirical attenuation relation. The magnitude saturation built into the constant stress drop single corner and double corner models is then constrained empirically, accommodating source finiteness in a manner consistent with the WUS strong motion database. This approach to limiting unrealistically high ground motions for large magnitude earthquakes at close distances is considered more physically reasonable than limiting the motions directly, which can be rather arbitrary and difficult to defend.

Because of the manner in which the model validations were performed ($\Delta\sigma$, Q(f), and H were optimized), parametric variability for only $\Delta\sigma$, Q(f) and H are required to be reflected in the model simulations (Appendix B; EPRI, 1993; Roblee et. al., 1996). For source depth variability, a lognormal distribution is used with a σ_{ln} = 0.6 (EPRI, 1993). Bounds are placed on the distribution to prevent nonphysical realizations (Table 2).

The stress drop variability, σ_{ln} = 0.7 is from EPRI (1993) and is based on inversions of ground motions for stress drop using CENA earthquakes. The variability in Q(f) is taken in Q_o alone (σ_{ln} = 0.4) and is based on inversions in WNA for Q(f) models. While not strictly required, crystalline basement kappa (0.006 sec) was also varied since its value is based entirely on data from other CENA regions and few CENA hard rock sites were available for the validation exercises (Silva et al., 1997). The variability for kappa (σ_{ln} =

0.3) is based on the variability seen in kappa values determined from strong ground motions recorded at about 20 Northern California rock sites which recorded the **M** 6.9 1989 Loma Prieta earthquake (EPRI, 1993).

While this uncertainty of 0.3 for kappa may seem low to characterize both epistemic (uncertainty in the median value) and aleatory (uncertainty about the median value) variability in a site specific kappa value, the point-source modeling uncertainty (Appendix A; Silva et al., 1997) already accommodates the effects of kappa variability. This arises because a fixed kappa value of 0.03 sec was used to characterize the linear rock damping at all rock sites in the validation exercises. As a result, site-specific departures of kappa values from the assumed value of 0.03 sec increase model departures from recorded motions resulting in larger estimates of model uncertainty. While it is possible that the total variability in the attenuation relations has been overestimated due to this probable double counting, validations are sparse for the CENA (nonexistent for deep soil sites) and for **M** larger than about 7.0 in the WNA. As a result, assessment and partition of appropriate variability is not an unambiguous issue, particularly in the CENA, and the approach taken here is to follow prudent design practice and not underestimate uncertainty.

Attenuation Relations

To generate data, which consists of 5% damped spectral acceleration, peak acceleration, peak particle velocity, and peak displacements, for the regression analyses, 300 simulations reflecting parametric variability are made at distances of 1, 5, 10, 20, 50, 75, 100, 200, and 400 km. At each distance, five magnitudes are used: **M** 4.5, 5.5, 6.5, 7.5, and 8.5 (Table 2).

The functional form selected for the regressions which provided the best overall fit (from a suite of about 25) to the simulations is given by

$$\ln y = C_1 + C_2\,\mathrm{M} + (C_6 + C_7\,\mathrm{M}) * \ln(R + e^{C_4}) + C_{10}\,(\mathrm{M} - 6)^2, \tag{3}$$

where R is taken as a closest distance to the surface projection of the rupture surface, consistent with the validation exercises (Silva et al., 1997).

Figure 1 shows the simulations for peak accelerations as well as the model fits for the single corner model with variable stress drops for **M** 7.5. In general, the model fits the central trends (medians) of the simulations. Figure 2 summarizes the magnitude dependency of the peak acceleration estimates and saturation is evident, primarily due to the magnitude dependent stress drop. Also evident is the magnitude dependent far-field fall off with a decrease in slope as **M** increases (easily seen beyond 100 km). This feature is especially important in the CEUS where large contributions to the hazard can come from distant sources. The model predicts peak accelerations at a distance of 1 km of about 0.30, 0.70, 1.10, 1.50g for **M** 4.5, 5.5, 6.5, and 7.5, respectively.

An example of response spectra at 1 km for **M** 4.5, 5.5, 6.5, 7.5, and 8.5 are shown in Figure 3. For **M** 7.5, the peak acceleration (Sa at 100 Hz) is about 1.8g with the peak in the spectrum near 0.04 sec. The jagged nature of the spectra is due to unsmoothed coefficients. The model regression coefficients are listed in Table 3 along with the parametric and total variability. The modeling variability is taken from Appendix A. The total variability, solid line in Figure 4, is large. It ranges from about 2 at short periods to about 4 at a period of 10 sec where it is dominated by modeling variability. This large long period uncertainty is due to the tendency of the point-source model to overpredict low frequency motions at large magnitudes (**M** > 6.5; EPRI, 1993). This trend led Atkinson and Silva (1997, 2000) to introduce a double-corner point-source model for WUS crustal sources, suggesting a similarity in source processes for WUS and CEUS crustal sources, but with CEUS sources being more energetic by about a factor of two (twice WUS stress drops), on average.

The results for the single corner frequency model with constant stress drop scaling are shown in Figures 5 to 8. The same plots are shown as were described for the previous model. These two models estimate similar values with the variable stress drop motions exceeding the constant stress drop motions at the lower magnitudes (**M** ≤ 6.5). The constant stress drop of 120 bars will result in about 30% to 50% higher rock motions at high frequency (> 1 Hz) for **M** 7.5 than the variable stress drop model, with a corresponding stress drop of 95 bars (EPRI, 1993). At small **M**, say **M** 5.5, the variable stress drop motions are higher, reflecting the 160 bar results of Atkinson for CEUS earthquakes with average **M** near 5.5. Also shown are the results for the model with saturation, reducing the large magnitude, close-in motions. The saturation reduces the **M** 7.5 and **M** 8.5 motions by 30 to 50% within about 10 km distance. The parametric variability is also similar to that of the variable stress drop model. The regression coefficients are given in Tables 4 and 5.

The regression results for the double corner frequency model are listed in Tables 6 and 7. The regression model fit to the peak acceleration data as shown in Figure 9. The PGA model is shown in Figure 10, and Figure 12 is a plot of the uncertainty. Figure 11 shows the spectra at a distance of 1 km. At long period (> 1 sec) and large **M** (≥ 6.5) the motions are significantly lower than those of the single-corner models (Figures 3 and 7). The parametric variability was taken as the same as the single corner model with variable stress drop as distributions are not currently available to apply to the two corner frequencies associated with this model (Atkinson and Boore, 1997). Since the two corner frequency source model was not available when the validations were performed (Silva et al., 1997), the model variability for the single corner frequency source model was used. This is considered conservative as the total variability for the two corner model is likely to be lower than that of the single corner model, as comparisons using WUS data show it provides a better fit to recorded motions at low frequencies (≤ 1 Hz; Atkinson and Silva, 1997, 2000). This is, of course, assuming the parametric variability associated with the two corner frequencies is not significantly larger than that associated with the single corner frequency stress drop.

At long period (> 1 sec) the total variability is largely empirical, being driven by the modeling component or comparisons to recorded motions. While this variability may be considered large, it includes about 17 earthquakes with magnitudes ranging from **M** 5.3

to **M** 7.4, distances out to 500 km, and both rock and soil sites. The average **M** for the validation earthquakes is about **M** 6.5, near the magnitude where empirical aleatory variability has a significant reduction (Abrahamson and Shedlock, 1997). The magnitude independent point-source variability may then reflect the generally higher variability associated with lower magnitude (**M** $\leq$ 6.5) earthquakes, being somewhat conservative for larger magnitude earthquakes. In view of all the uncertainties present in estimating strong ground motions in the CENA, the total variability estimates, although unpleasant, are probably realistic and reflect the substantial current lack of knowledge in addition to randomness.

It is important to emphasize that although the total variability is considered appropriate for the CEUS, it is assumed to be totally aleatory or randomness in nature. Epistemic variability or uncertainty in mean estimates of ground motions is assumed to be totally accommodated in the use of the three models with appropriate weights. These assumptions may underpredict epistemic variability if indeed the extremely high stress drop of the 1988 **M** 5.8 Saquenay, Canada earthquake reflects a different stress drop population. In that case one should develop low, medium, and high stress drop attenuation relations for each of the three source models. As a consequence, the stress drop variability (comprised of both epistemic and aleatory components) of σ_{ln} = 0.7 (Table 2) would be reduced to a value near 0.5, or even less for the magnitude dependent stress drop model, based on inversions of WUS earthquakes (Silva et al., 1997). With this approach the total variability may remain largely unchanged but fractile estimates of hazard curves would reflect the assumed partition of aleatory and epistemic variability in CEUS stress drops.

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Table 1 CRUSTAL MODEL*		
Thickness (km)	V_S (km/sec)	Density (cgs)
1.30	2.83	2.52
11.00	3.52	2.71
28.00	3.75	2.78
-----	4.62	3.35

* EPRI mid-continent (EPRI, 1993; Toro et al., 1997)

Table 2 PARAMETERS FOR CRYSTALLINE ROCK OUTCROP ATTENUATION SIMULATIONS				
M	4.5, 5.5, 6.5, 7.5, 8.5			
D (km)	1, 5, 10, 20, 50, 75, 100, 200, 400			
300 simulations for each **M**, *R* pair				
Randomly vary source depth, $\Delta\sigma$, kappa, Q_o, η, profile				
<u>Depth,</u> $\sigma_{\ln H} = 0.6$, Intraplate Seismicity (EPRI, 1993)				
M	m_{blg}	Lower Bound (km)	$\overline{H}$ (km)	Upper Bound (km)
4.5	4.9	2	6	15
5.5	6.0	2	6	15
6.5	6.6	4	8	20
7.5	7.1	5	10	20
8.5	7.8	5	10	20
$\Delta\sigma$, $\sigma_{\ln \Delta\sigma} = 0.7$ (EPRI, 1993)				
M	m_{blg}	$\Delta\sigma$ (bars)	AVG. $\Delta\sigma$ (bars) = 123; Assumes **M** 5.5 = 160 bars (Atkinson, 1993) with magnitude scaling taken from WUS (Silva et al., 1997); constant stress drop model has $\Delta\sigma$ (bars) = 120	
4.5	4.9	160, 120*		
5.5	6.0	160, 120*		
6.5	6.6	120, 120*		
7.5	7.1	90, 120*		
8.5	7.8	70, 120*		
<u>Q(s),</u> $\overline{Q}_o = 351$, (Silva et al., 1997) $\sigma_{\ln Q_o} = 0.4$, (Silva et al., 1997) $\eta = 0.84$, (Silva et al., 1997), $\sigma_\eta = 0$, (Silva et al., 1997) Varying Q_o only sufficient, ± 1 σ covers range of CEUS inversions from 1 to 20 Hz				
Kappa, $\overline{\kappa} = 0.006$ sec; $\sigma_{\ln\kappa} = 0.3$, (EPRI, 1993)				
<u>Profile,</u> Crystaline Basement, randomize top 100 ft				
Geometrical attenuation $R^{-(a + b(\mathbf{M} - 6.5))}$, a = 1.0296, b = -0.0422 $R^{-(a + b(\mathbf{M} - 6.5))/2}$, R > 80 km, approximately twice crustal thickness (Table1) Based on inversions of the Abrahamson and Silva (1997) relation				

*Constant Stress Drop Model

Table 3
HARD ROCK
REGRESSION COEFFICIENTS FOR THE SINGLE CORNER MODEL WITH
VARIABLE STRESS DROP AS A FUNCTION OF MOMENT MAGNITUDE (**M**)

Freq. Hz	C1	C2	C4	C5	C6	C7	C8	C10	Parametric Sigma	Total Sigma
0.1000	-18.88369	2.53845	2.10000	.00000	-1.44182	.05839	.00000	-.30968	.4199	1.3429
0.2000	-14.96510	2.23977	2.30000	.00000	-1.58733	.06610	.00000	-.38601	.4529	1.2228
0.3333	-11.66550	1.92782	2.40000	.00000	-1.73789	.07805	.00000	-.39560	.4887	1.0871
0.5000	-8.82667	1.63766	2.50000	.00000	-1.89973	.09204	.00000	-.37384	.5217	1.0095
0.6250	-7.43948	1.47547	2.50000	.00000	-1.97181	.09962	.00000	-.35266	.5388	.9450
1.0000	-4.35940	1.10344	2.60000	.00000	-2.19556	.12077	.00000	-.29213	.5697	.8739
1.3333	-2.66885	.90369	2.60000	.00000	-2.28860	.13067	.00000	-.24886	.5844	.8815
2.0000	-.70065	.66331	2.60000	.00000	-2.42230	.14517	.00000	-.19572	.6016	.8426
2.5000	.27922	.55253	2.60000	.00000	-2.48052	.15109	.00000	-.16795	.6110	.8320
3.3333	1.27630	.43069	2.60000	.00000	-2.56373	.15989	.00000	-.14085	.6231	.8358
4.1667	1.94719	.36067	2.60000	.00000	-2.61226	.16463	.00000	-.12422	.6326	.8272
5.0000	2.43129	.31603	2.60000	.00000	-2.64479	.16751	.00000	-.11332	.6412	.8260
6.2500	3.29848	.24288	2.70000	.00000	-2.75925	.17594	.00000	-.10319	.6535	.8290
6.6667	3.42429	.23007	2.70000	.00000	-2.77224	.17711	.00000	-.10076	.6587	.8339
8.3333	3.82611	.19671	2.70000	.00000	-2.80768	.18000	.00000	-.09412	.6717	.8485
10.0000	4.09661	.17267	2.70000	.00000	-2.83744	.18244	.00000	-.09012	.6837	.8468
12.5000	4.36936	.14791	2.70000	.00000	-2.87191	.18520	.00000	-.08661	.6923	.8476
14.2857	4.49895	.13453	2.70000	.00000	-2.89249	.18688	.00000	-.08509	.6982	.8521
16.6667	5.06492	.09382	2.80000	.00000	-2.99306	.19346	.00000	-.08376	.7058	.8602
18.1818	5.13193	.08668	2.80000	.00000	-3.00574	.19447	.00000	-.08317	.7106	.8604
20.0000	5.20282	.07947	2.80000	.00000	-3.01917	.19552	.00000	-.08262	.7165	.8675
25.0000	5.37048	.06513	2.80000	.00000	-3.04845	.19764	.00000	-.08164	.7339	.8795
31.0000	5.53534	.05373	2.80000	.00000	-3.07478	.19936	.00000	-.08092	.7462	.8869
40.0000	6.17831	.01178	2.90000	.00000	-3.19033	.20660	.00000	-.08055	.7534	.8902
50.0000	6.21252	.00220	2.90000	.00000	-3.21297	.20863	.00000	-.08080	.7561	.8939
100.000	4.39500	.06737	2.70000	.00000	-3.02023	.20242	.00000	-.08804	.6994	.8468
PGA	4.19301	.07506	2.70000	.00000	-3.00408	.20195	.00000	-.08927	.6912	.8400
PGV	3.39476	.62991	2.40000	.00000	-2.76262	.20554	.00000	-.13908	.5582	------

Table 4
HARD ROCK
REGRESSION COEFFICIENTS FOR THE SINGLE CORNER MODEL WITH CONSTANT STRESS DROP

Freq. Hz	C1	C2	C4	C5	C6	C7	C8	C10	Parametric Sigma	Total Sigma
0.1000	-19.28945	2.59965	2.10000	.00000	-1.43865	.05802	.00000	-.26815	.4106	1.3400
0.2000	-15.41671	2.31029	2.30000	.00000	-1.58264	.06531	.00000	-.34272	.4483	1.2211
0.3333	-12.14626	2.00333	2.40000	.00000	-1.73224	.07698	.00000	-.35126	.4854	1.0857
0.5000	-9.33586	1.71687	2.50000	.00000	-1.89335	.09080	.00000	-.32809	.5177	1.0074
0.6250	-7.97407	1.55786	2.50000	.00000	-1.96423	.09820	.00000	-.30626	.5340	.9423
1.0000	-4.96496	1.19548	2.60000	.00000	-2.18581	.11900	.00000	-.24566	.5639	.8701
1.3333	-3.32213	1.00226	2.60000	.00000	-2.27821	.12880	.00000	-.20311	.5787	.8777
2.0000	-1.43548	.77358	2.60000	.00000	-2.40928	.14297	.00000	-.15233	.5971	.8394
2.5000	-.50467	.67008	2.60000	.00000	-2.46619	.14872	.00000	-.12648	.6075	.8295
3.3333	.43668	.55655	2.60000	.00000	-2.54754	.15729	.00000	-.10162	.6204	.8338
4.1667	1.06963	.49226	2.60000	.00000	-2.59470	.16186	.00000	-.08664	.6304	.8255
5.0000	1.52694	.45169	2.60000	.00000	-2.62629	.16462	.00000	-.07695	.6393	.8246
6.2500	2.36181	.38344	2.70000	.00000	-2.73859	.17274	.00000	-.06802	.6516	.8275
6.6667	2.47996	.37179	2.70000	.00000	-2.75112	.17386	.00000	-.06588	.6568	.8324
8.3333	2.86034	.34166	2.70000	.00000	-2.78522	.17656	.00000	-.06010	.6698	.8470
10.0000	3.11592	.31985	2.70000	.00000	-2.81384	.17885	.00000	-.05663	.6817	.8452
12.5000	3.37375	.29732	2.70000	.00000	-2.84698	.18144	.00000	-.05357	.6902	.8459
14.2857	3.92488	.25884	2.80000	.00000	-2.94339	.18762	.00000	-.05225	.6961	.8503
16.6667	4.05039	.24603	2.80000	.00000	-2.96583	.18937	.00000	-.05110	.7036	.8584
18.1818	4.11372	.23943	2.80000	.00000	-2.97807	.19032	.00000	-.05058	.7084	.8586
20.0000	4.18102	.23274	2.80000	.00000	-2.99106	.19131	.00000	-.05011	.7143	.8657
25.0000	4.34191	.21938	2.80000	.00000	-3.01949	.19332	.00000	-.04924	.7316	.8776
31.0000	4.50147	.20875	2.80000	.00000	-3.04513	.19494	.00000	-.04861	.7439	.8849
40.0000	5.13460	.16824	2.90000	.00000	-3.15909	.20195	.00000	-.04828	.7511	.8883
50.0000	5.16438	.15928	2.90000	.00000	-3.18078	.20386	.00000	-.04851	.7537	.8918
100.000	3.36079	.22218	2.70000	.00000	-2.98760	.19761	.00000	-.05509	.6969	.8447
PGA	3.16202	.22938	2.70000	.00000	-2.97147	.19714	.00000	-.05620	.6886	.8379
PGV	2.51086	.76168	2.40000	.00000	-2.72601	.20021	.00000	-.10368	.5550	------

Table 5 HARD ROCK REGRESSION COEFFICIENTS FOR THE SINGLE CORNER MODEL WITH CONSTANT STRESS DROP AND SATURATION										
Freq. Hz	C1	C2	C4	C5	C6	C7	C8	C10	Parametric Sigma	Total Sigma
0.1000	-17.69763	2.33877	2.30000	.00000	-1.75359	.11071	.00000	-.28005	.4204	1.3431
0.2000	-13.69697	2.03488	2.50000	.00000	-1.91969	.12052	.00000	-.35463	.4597	1.2253
0.3333	-10.33313	1.71755	2.60000	.00000	-2.08560	.13401	.00000	-.36316	.4981	1.0914
0.5000	-7.42051	1.41946	2.70000	.00000	-2.26433	.14984	.00000	-.33999	.5309	1.0142
0.6250	-6.03692	1.25821	2.70000	.00000	-2.33925	.15765	.00000	-.31816	.5472	.9498
1.0000	-2.89906	.88116	2.80000	.00000	-2.58296	.18098	.00000	-.25757	.5767	.8785
1.3333	-1.22930	.68518	2.80000	.00000	-2.68016	.19127	.00000	-.21501	.5914	.8861
2.0000	.69539	.45254	2.80000	.00000	-2.81800	.20613	.00000	-.16423	.6097	.8484
2.5000	1.64228	.34751	2.80000	.00000	-2.87774	.21215	.00000	-.13838	.6199	.8386
3.3333	2.60689	.23165	2.80000	.00000	-2.96321	.22112	.00000	-.11352	.6325	.8428
4.1667	3.25319	.16616	2.80000	.00000	-3.01272	.22589	.00000	-.09854	.6423	.8346
5.0000	3.71953	.12490	2.80000	.00000	-3.04591	.22877	.00000	-.08886	.6511	.8338
6.2500	4.18416	.07875	2.80000	.00000	-3.09159	.23297	.00000	-.07992	.6633	.8368
6.6667	4.30417	.06695	2.80000	.00000	-3.10445	.23411	.00000	-.07779	.6685	.8417
8.3333	5.17320	.00188	2.90000	.00000	-3.22475	.24286	.00000	-.07200	.6814	.8562
10.0000	5.43782	-.02059	2.90000	.00000	-3.25499	.24527	.00000	-.06853	.6933	.8546
12.5000	5.70631	-.04389	2.90000	.00000	-3.29005	.24799	.00000	-.06548	.7017	.8553
14.2857	5.83477	-.05663	2.90000	.00000	-3.31101	.24965	.00000	-.06416	.7077	.8599
16.6667	5.96397	-.06970	2.90000	.00000	-3.33411	.25145	.00000	-.06300	.7152	.8680
18.1818	6.56761	-.11492	3.00000	.00000	-3.44099	.25904	.00000	-.06249	.7200	.8682
20.0000	6.63937	-.12193	3.00000	.00000	-3.45478	.26008	.00000	-.06201	.7259	.8753
25.0000	6.81012	-.13594	3.00000	.00000	-3.48499	.26220	.00000	-.06115	.7429	.8870
31.0000	6.97878	-.14713	3.00000	.00000	-3.51228	.26392	.00000	-.06051	.7550	.8943
40.0000	7.14087	-.15876	3.00000	.00000	-3.54274	.26589	.00000	-.06019	.7619	.8974
50.0000	7.17445	-.16806	3.00000	.00000	-3.56511	.26786	.00000	-.06041	.7643	.9008
100.000	5.73885	-.12424	2.90000	.00000	-3.43887	.26510	.00000	-.06699	.7079	.8538
PGA	5.53459	-.11691	2.90000	.00000	-3.42173	.26461	.00000	-.06810	.6998	.8471
PGV	4.60099	.44166	2.60000	.00000	-3.13013	.26350	.00000	-.11559	.5648	--------

Table 6 HARD ROCK REGRESSION COEFFICIENTS FOR THE DOUBLE CORNER MODEL										
Freq. Hz	C1	C2	C4	C5	C6	C7	C8	C10	Parametric Sigma	Total Sigma
0.1000	-17.74463	2.22485	2.10000	.00000	-1.40084	.05305	.00000	-.31641	.4199	1.3429
0.2000	-13.88893	1.89859	2.30000	.00000	-1.54772	.06068	.00000	-.28960	.4529	1.2228
0.3333	-11.04809	1.64665	2.40000	.00000	-1.70010	.07272	.00000	-.22943	.4887	1.0871
0.5000	-8.76880	1.45200	2.50000	.00000	-1.86494	.08722	.00000	-.18125	.5217	1.0095
0.6250	-7.68301	1.34978	2.50000	.00000	-1.94573	.09603	.00000	-.16127	.5388	.9450
1.0000	-5.47019	1.12590	2.50000	.00000	-2.13473	.11710	.00000	-.13830	.5697	.8739
1.3333	-3.77355	.98718	2.60000	.00000	-2.28113	.13007	.00000	-.13323	.5844	.8815
2.0000	-1.95968	.80810	2.60000	.00000	-2.41132	.14449	.00000	-.12529	.6016	.8426
2.5000	-.96872	.71370	2.60000	.00000	-2.46500	.15003	.00000	-.11749	.6110	.8320
3.3333	.10920	.59537	2.60000	.00000	-2.54120	.15808	.00000	-.10506	.6231	.8358
4.1667	.86777	.52085	2.60000	.00000	-2.58506	.16235	.00000	-.09484	.6326	.8272
5.0000	1.42831	.46988	2.60000	.00000	-2.61380	.16486	.00000	-.08671	.6412	.8260
6.2500	1.99361	.41219	2.60000	.00000	-2.65510	.16868	.00000	-.07801	.6535	.8290
6.6667	2.14018	.39715	2.60000	.00000	-2.66676	.16973	.00000	-.07573	.6587	.8339
8.3333	2.60454	.35667	2.60000	.00000	-2.69927	.17238	.00000	-.06929	.6717	.8485
10.0000	3.30684	.30373	2.70000	.00000	-2.79751	.17893	.00000	-.06512	.6837	.8468
12.5000	3.62400	.27369	2.70000	.00000	-2.83163	.18170	.00000	-.06128	.6923	.8476
14.2857	3.77510	.25773	2.70000	.00000	-2.85226	.18339	.00000	-.05952	.6982	.8521
16.6667	3.92454	.24169	2.70000	.00000	-2.87495	.18521	.00000	-.05791	.7058	.8602
18.1818	3.99907	.23357	2.70000	.00000	-2.88734	.18619	.00000	-.05717	.7106	.8604
20.0000	4.07670	.22547	2.70000	.00000	-2.90040	.18720	.00000	-.05647	.7165	.8675
25.0000	4.69293	.18262	2.80000	.00000	-3.00672	.19396	.00000	-.05520	.7339	.8795
31.0000	4.86717	.17018	2.80000	.00000	-3.03252	.19560	.00000	-.05434	.7462	.8869
40.0000	5.03119	.15779	2.80000	.00000	-3.06134	.19746	.00000	-.05377	.7534	.8902
50.0000	5.06834	.14806	2.80000	.00000	-3.08409	.19935	.00000	-.05361	.7561	.8939
100.000	3.74623	.18152	2.70000	.00000	-2.98867	.19854	.00000	-.05734	.6994	.8468
PGA	3.54103	.18904	2.70000	.00000	-2.97418	.19819	.00000	-.05814	.6912	.8400
PGV	4.06989	.46794	2.50000	.00000	-2.77481	.19743	.00000	-.07606	.5582	--------

NOTE: PARAMETRIC SIGMA VALUES ARE FROM THE 1 CORNER VARIABLE STRESS DROP MODEL

Table 7 HARD ROCK REGRESSION COEFFICIENTS FOR THE DOUBLE CORNER MODEL WITH SATURATION										
Freq. Hz	C1	C2	C4	C5	C6	C7	C8	C10	Parametric Sigma	Total Sigma
0.1000	-16.16329	1.96535	2.30000	.00000	-1.71374	.10547	.00000	-.32832	.4199	1.3429
0.2000	-12.17910	1.62451	2.50000	.00000	-1.88291	.11564	.00000	-.30150	.4529	1.2228
0.3333	-9.24347	1.36201	2.60000	.00000	-2.05193	.12954	.00000	-.24133	.4887	1.0871
0.5000	-6.86049	1.15548	2.70000	.00000	-2.23472	.14610	.00000	-.19315	.5217	1.0095
0.6250	-5.75016	1.05061	2.70000	.00000	-2.32003	.15540	.00000	-.17317	.5388	.9450
1.0000	-3.10841	.79561	2.80000	.00000	-2.58562	.18195	.00000	-.15020	.5697	.8739
1.3333	-1.68010	.66971	2.80000	.00000	-2.68318	.19261	.00000	-.14513	.5844	.8815
2.0000	.17104	.48663	2.80000	.00000	-2.81997	.20773	.00000	-.13719	.6016	.8426
2.5000	1.17695	.39078	2.80000	.00000	-2.87626	.21352	.00000	-.12940	.6110	.8320
3.3333	2.27626	.27031	2.80000	.00000	-2.95623	.22193	.00000	-.11697	.6231	.8358
4.1667	3.04705	.19471	2.80000	.00000	-3.00223	.22639	.00000	-.10675	.6326	.8272
5.0000	3.61568	.14311	2.80000	.00000	-3.03239	.22900	.00000	-.09861	.6412	.8260
6.2500	4.19281	.08441	2.80000	.00000	-3.07579	.23300	.00000	-.08991	.6535	.8290
6.6667	4.34277	.06911	2.80000	.00000	-3.08805	.23409	.00000	-.08764	.6587	.8339
8.3333	4.81663	.02793	2.80000	.00000	-3.12224	.23686	.00000	-.08119	.6717	.8485
10.0000	5.13706	-.00173	2.80000	.00000	-3.15185	.23929	.00000	-.07703	.6837	.8468
12.5000	5.94942	-.06741	2.90000	.00000	-3.27328	.24822	.00000	-.07318	.6923	.8476
14.2857	6.10708	-.08387	2.90000	.00000	-3.29509	.25000	.00000	-.07142	.6982	.8521
16.6667	6.26384	-.10044	2.90000	.00000	-3.31911	.25192	.00000	-.06982	.7058	.8602
18.1818	6.34238	-.10886	2.90000	.00000	-3.33222	.25295	.00000	-.06908	.7106	.8604
20.0000	6.42423	-.11726	2.90000	.00000	-3.34604	.25401	.00000	-.06838	.7165	.8675
25.0000	6.61204	-.13370	2.90000	.00000	-3.37593	.25613	.00000	-.06711	.7339	.8795
31.0000	7.33736	-.18563	3.00000	.00000	-3.49824	.26456	.00000	-.06625	.7462	.8869
40.0000	7.51145	-.19862	3.00000	.00000	-3.52888	.26652	.00000	-.06568	.7534	.8902
50.0000	7.55648	-.20898	3.00000	.00000	-3.55306	.26853	.00000	-.06551	.7561	.8939
100.000	6.12213	-.16489	2.90000	.00000	-3.43941	.26601	.00000	-.06925	.6994	.8468
PGA	5.91196	-.15727	2.90000	.00000	-3.42401	.26564	.00000	-.07004	.6912	.8400
PGV	5.79531	.17529	2.60000	.00000	-3.11215	.25573	.00000	-.08796	.5582	--------

NOTE: PARAMETRIC SIGMA VALUES ARE FROM THE 1 CORNER VARIABLE STRESS DROP MODEL

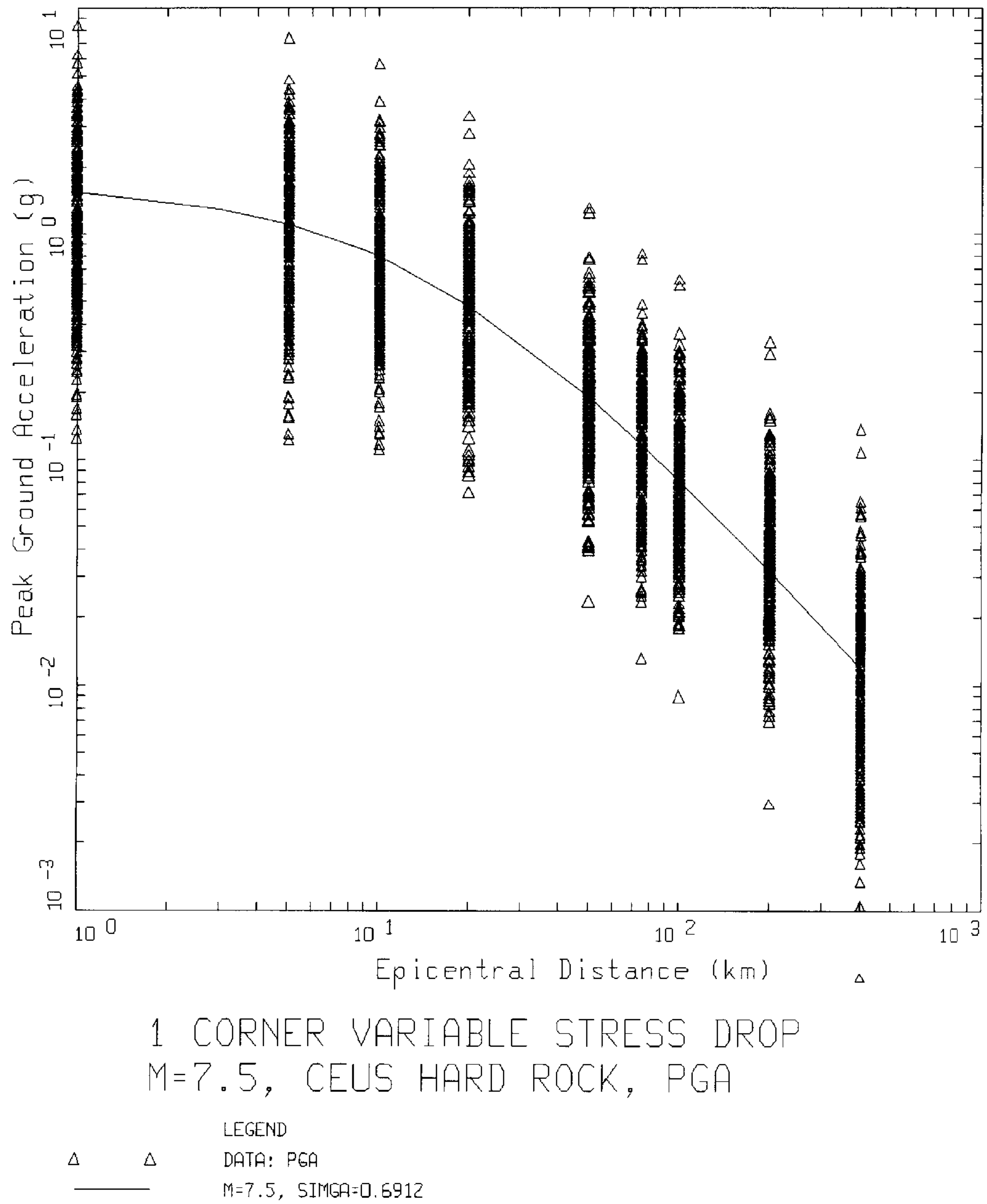

Figure 1. Peak acceleration estimates and regression fit at **M** 7.5 for the single corner model with variable stress drop.

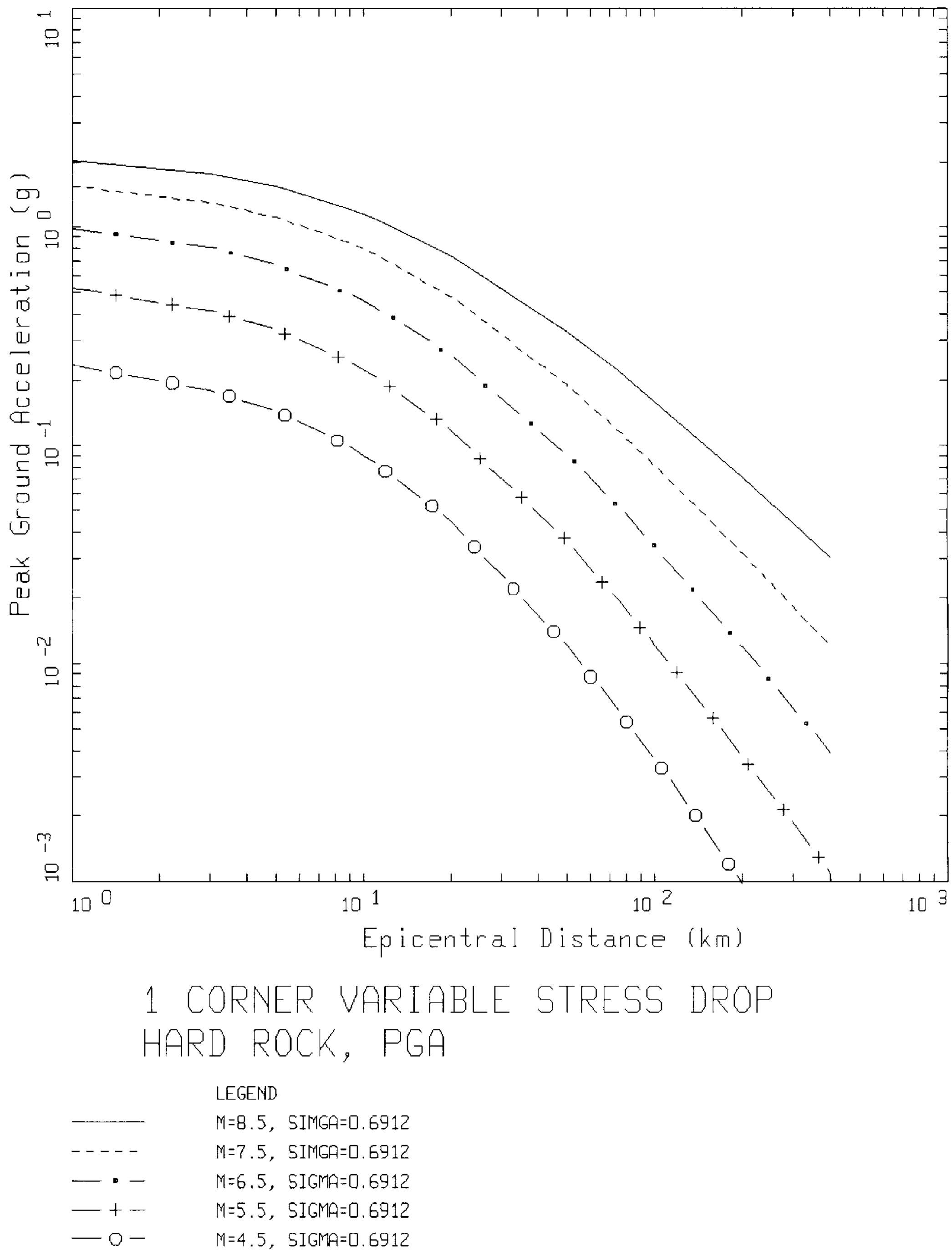

Figure 2. Attenuation of median peak horizontal accelerations at **M** 4.5, 5.5, 6.5, 7.5 and 8.5 for the single corner model with variable stress drop.

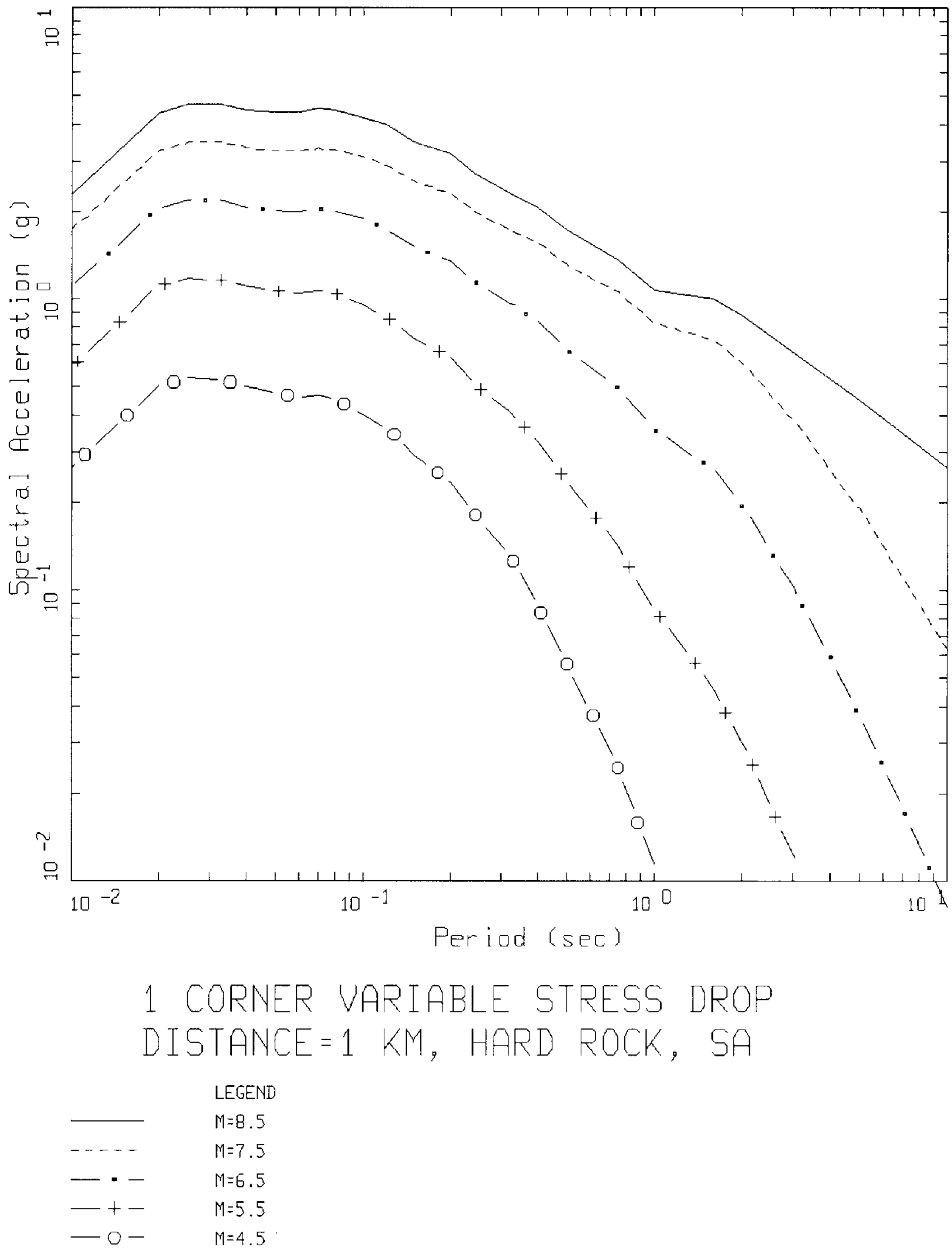

Figure 3. Median response spectra (5% damping) at a distance of 1 km for magnitudes **M** 4.5, 5.5, 6.5, 7.5, and 8.5 for the single corner model with variable stress drop.

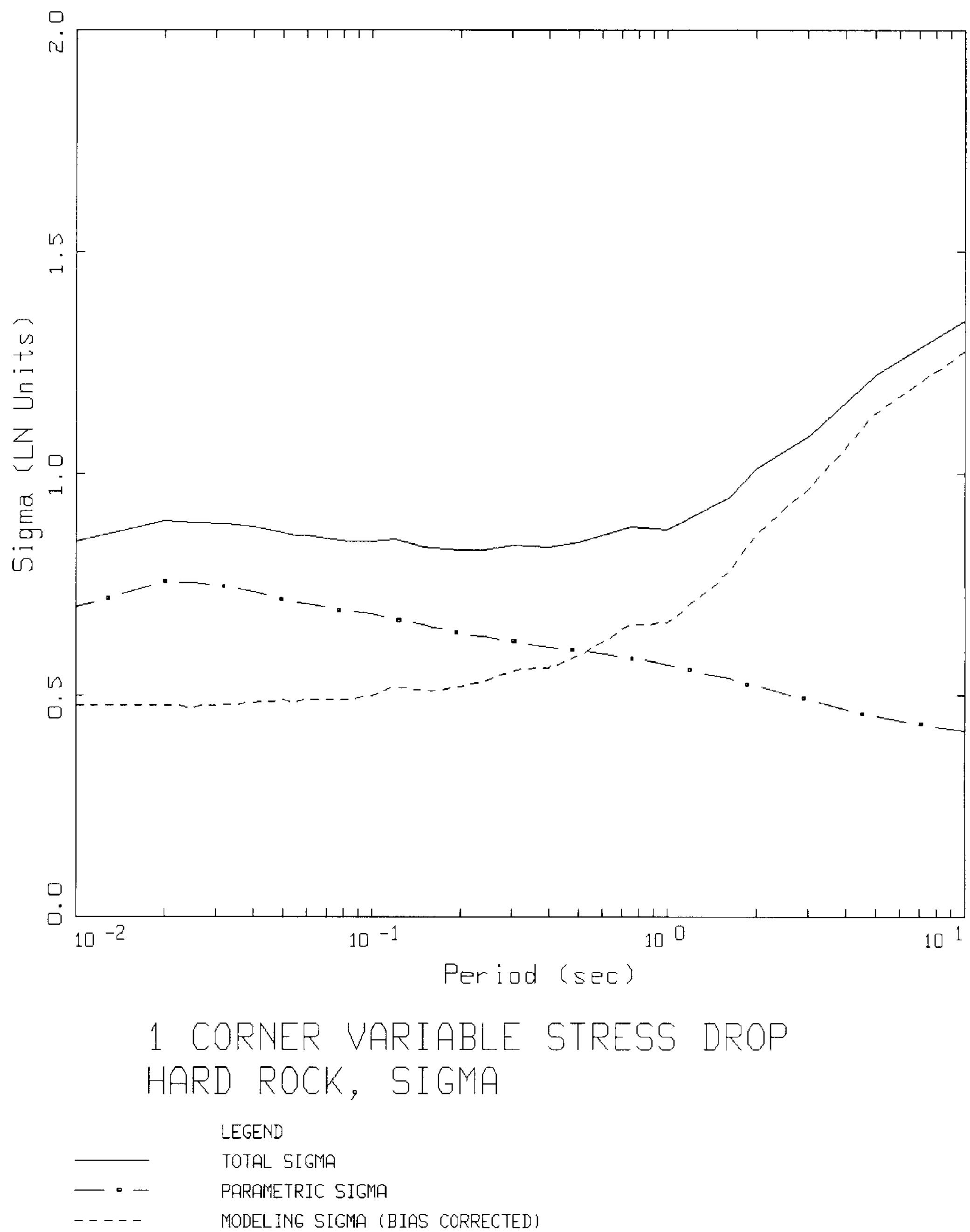

Figure 4. Estimates of total variability (uncertainty) for the attenuation model. Parametric variability is due to variation of variable stress drop, single corner frequency point-source parameters (Table 2), and fit of regression model (Table 3). Model variability is from validation exercises with 16 earthquakes (**M** 5.3 to 7.4) at 500 sites over the fault distance range of 1 to 460 km (Appendix B).

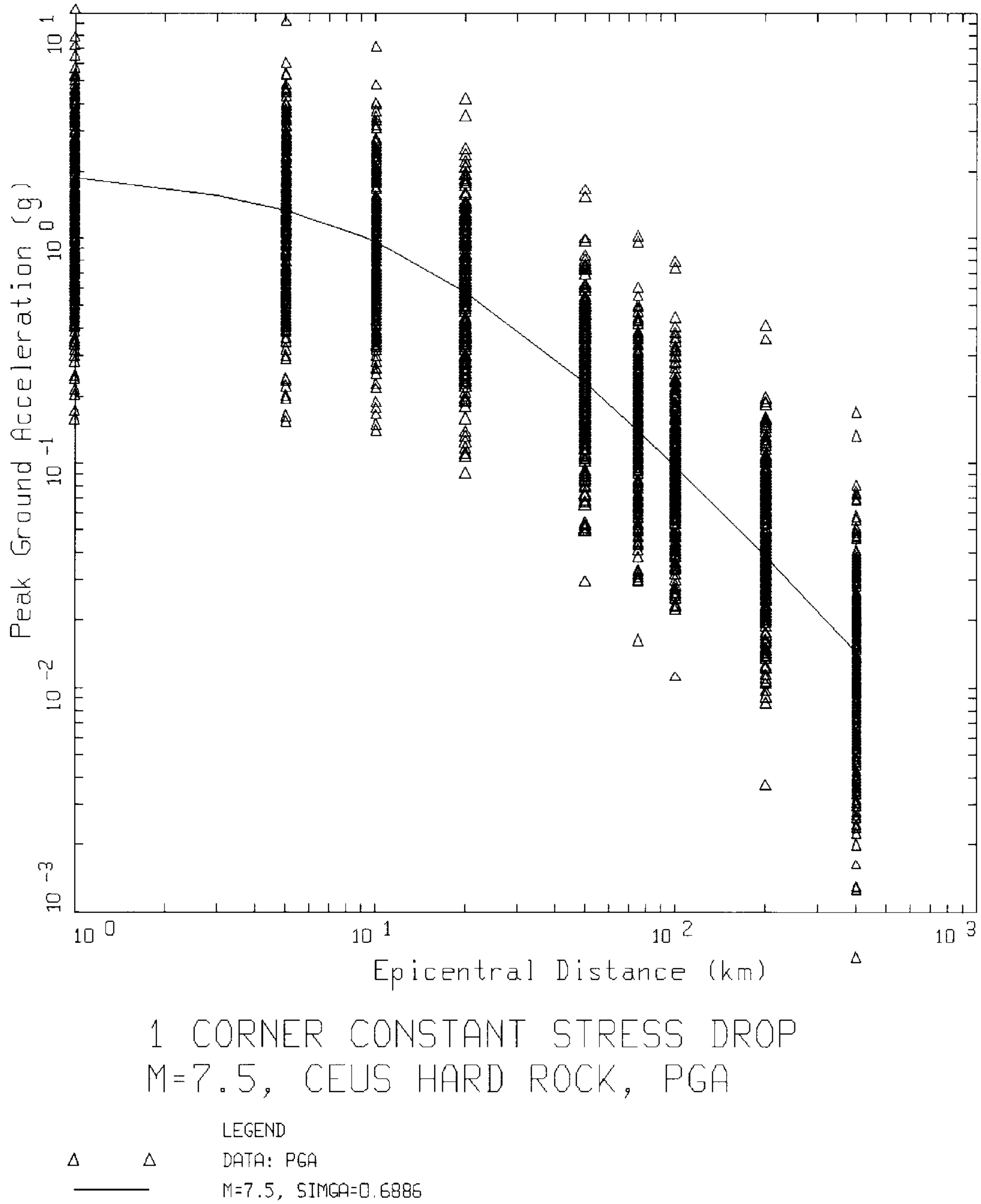

Figure 5. Peak acceleration estimates and regression fit at **M** 7.5 for the single corner model with constant stress drop.

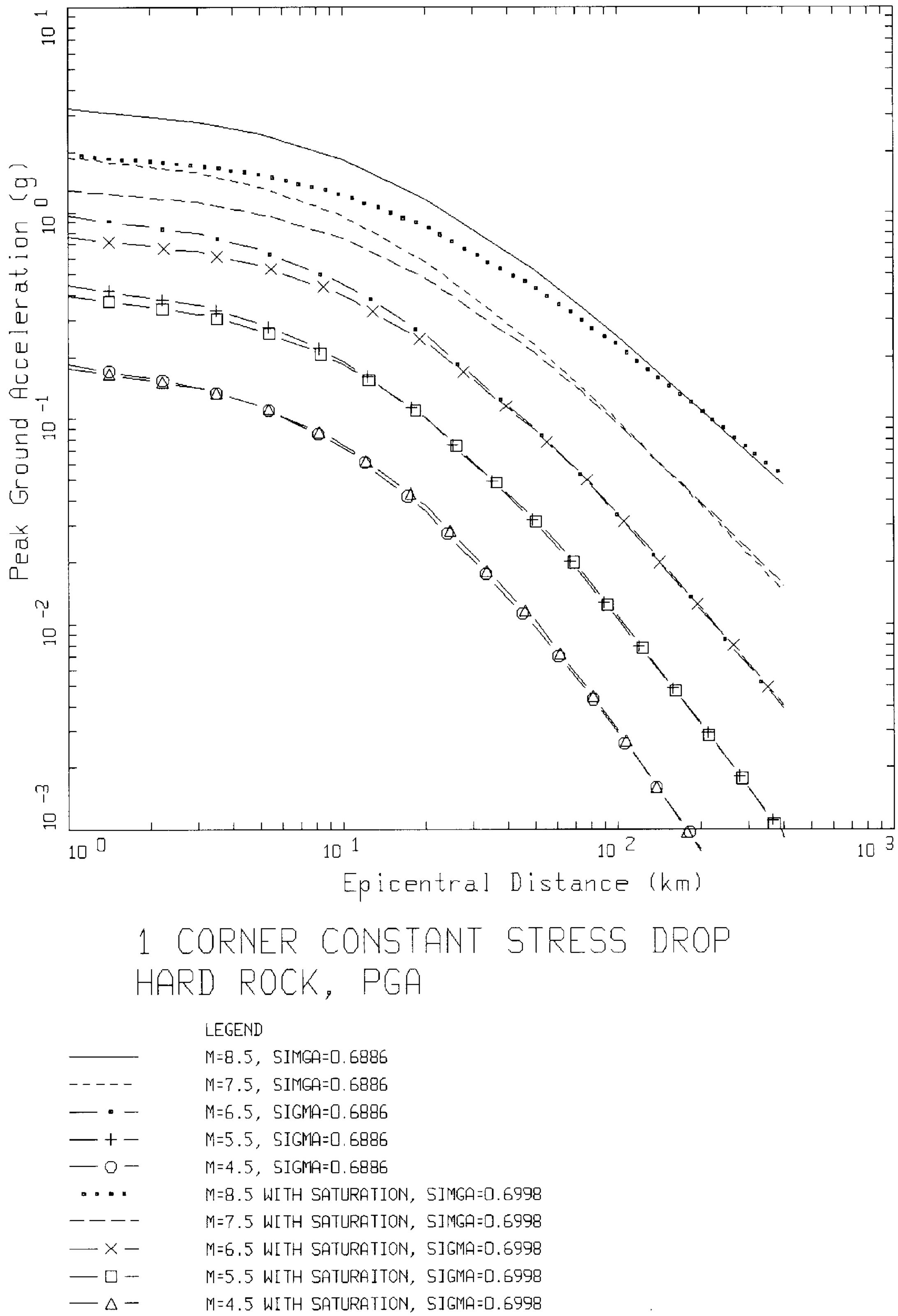

Figure 6. Attenuation of median peak horizontal accelerations at **M** 4.5, 5.5, 6.5, 7.5, and 8.5 for the single corner model with constant stress drop, with and without saturation.

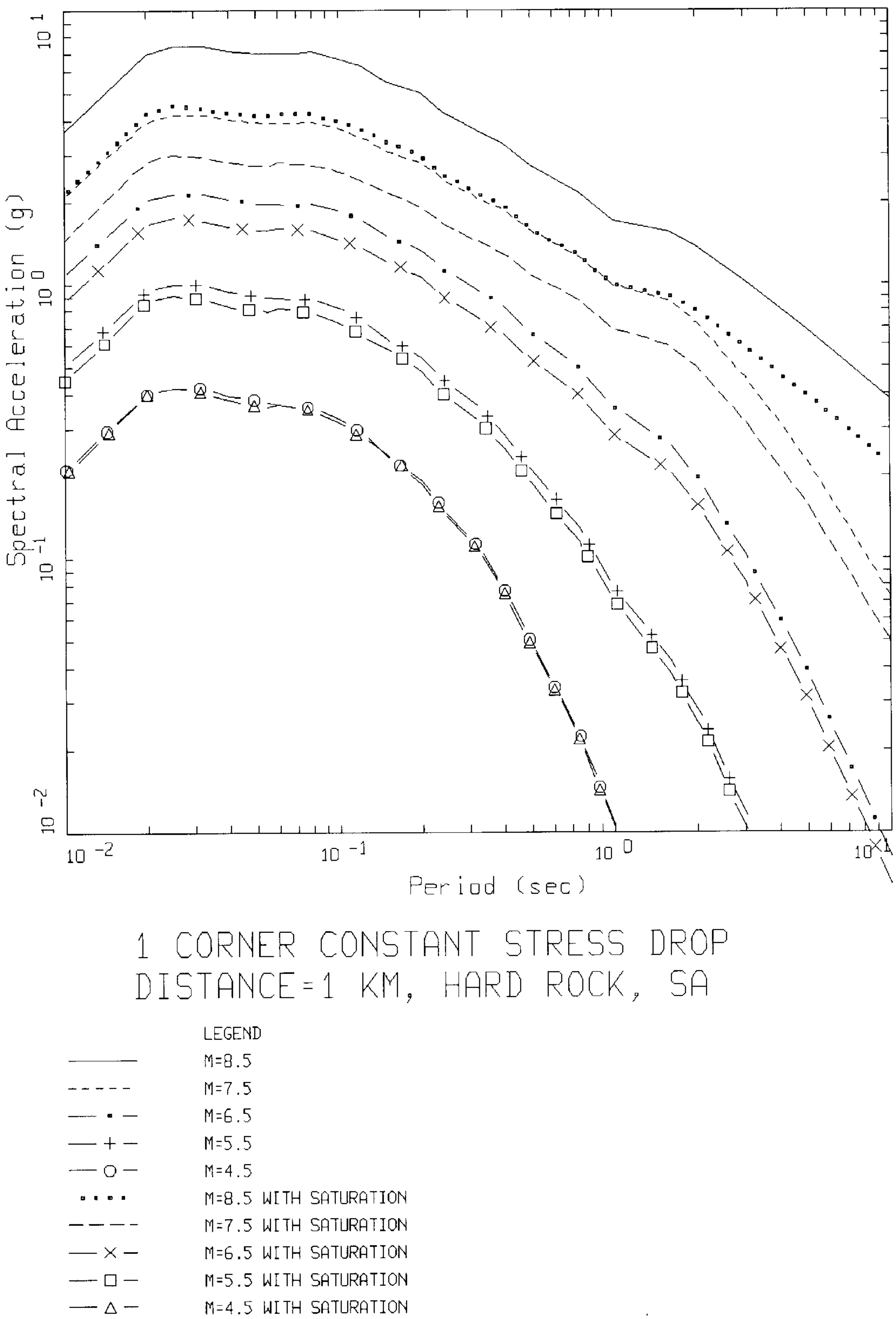

Figure 7. Median response spectra (5% damping) at a distance of 1 km for magnitudes **M** 4.5, 5.5, 6.5, 7.5, and 8.5 for the single corner model with constant stress drop, with and without saturation.

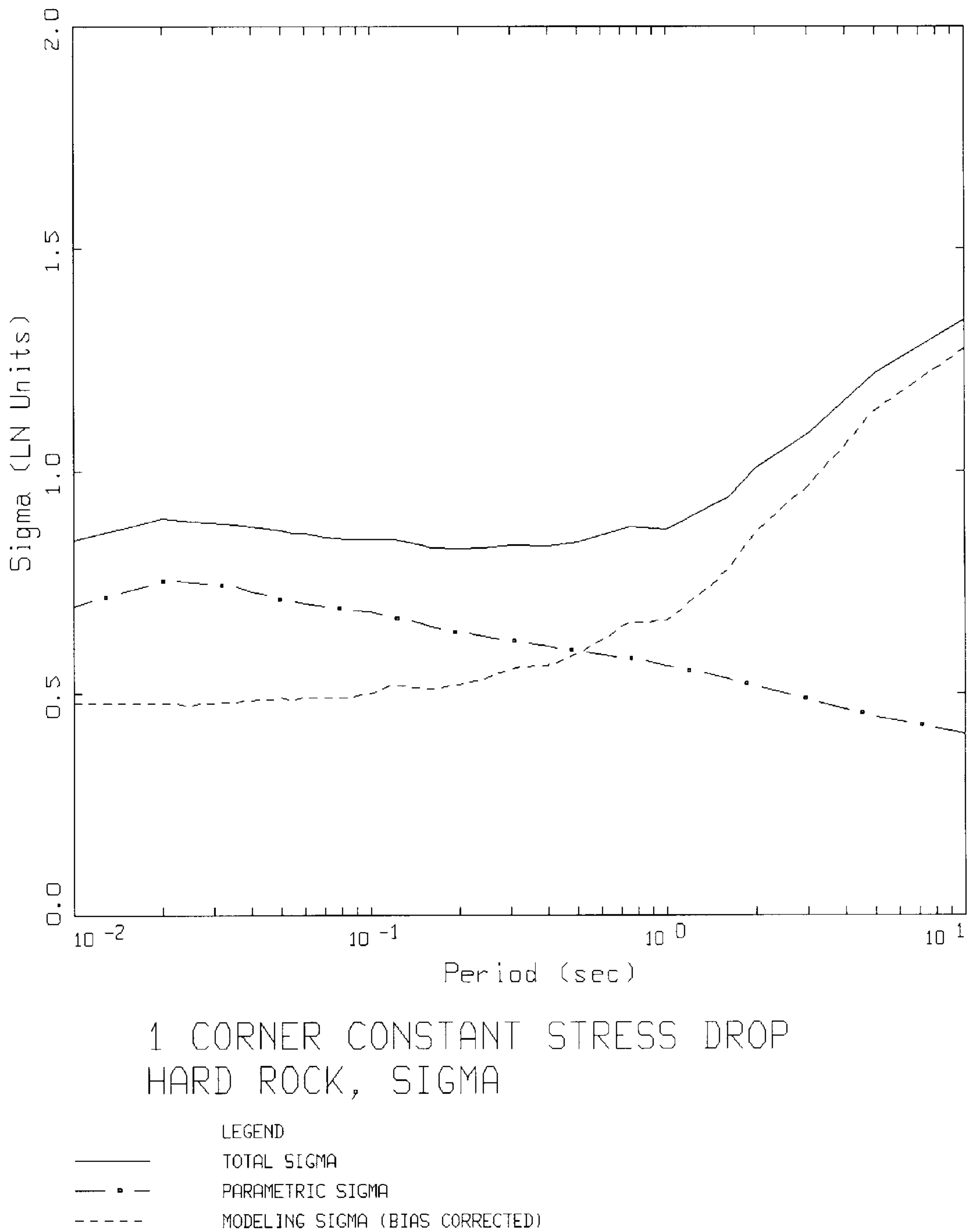

Figure 8. Estimates of total variability (uncertainty) for the attenuation model. Parametric variability is due to variation of constant stress drop, single corner frequency point-source parameters (Table 2), and fit of regression model (Table 4). Model variability is from validation exercises with 16 earthquakes (**M** 5.3 to 7.4) at 500 sites over the fault distance range of 1 to 460 km (Appendix B).

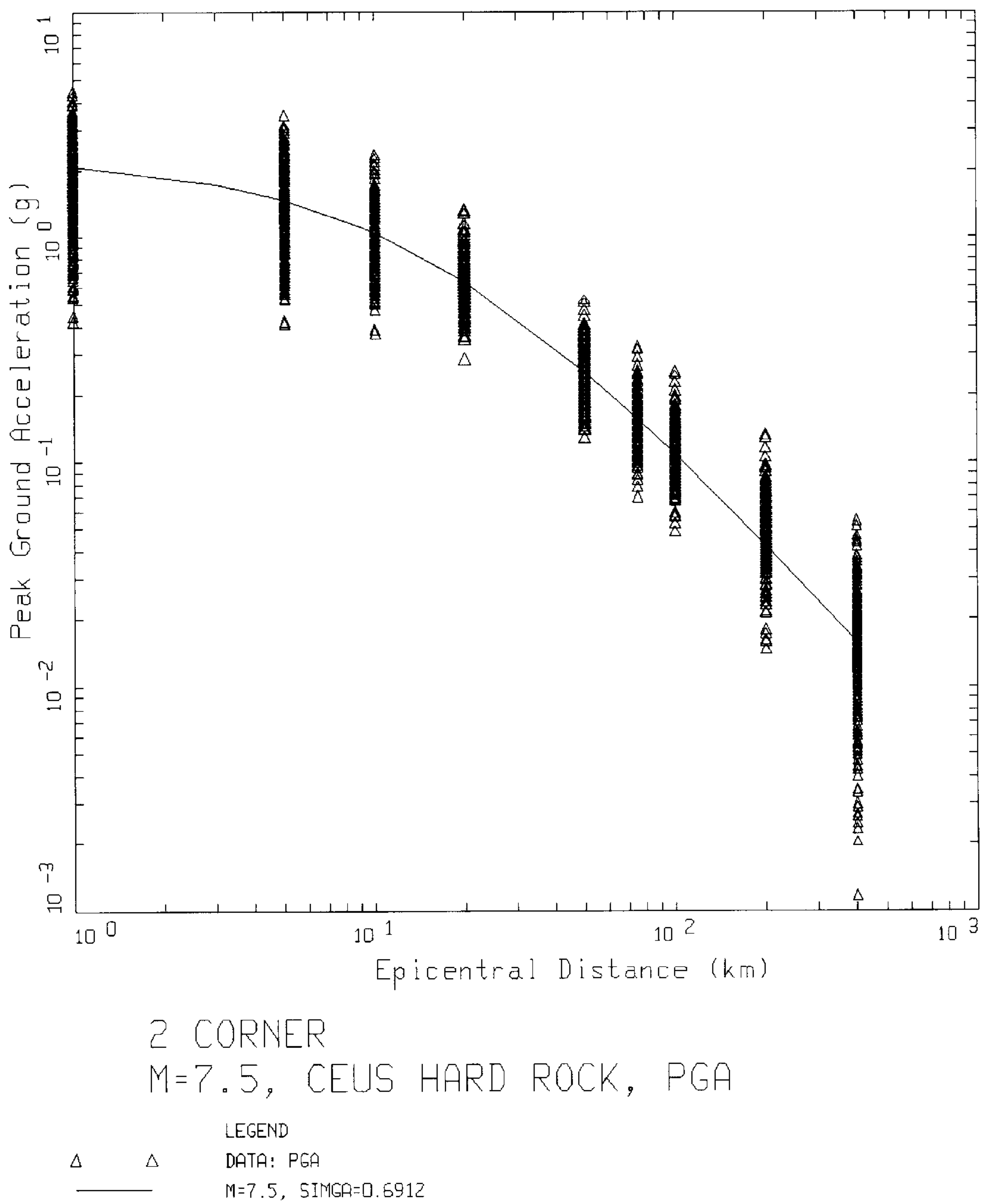

Figure 9. Peak acceleration estimates and regression fit at **M** 7.5 for the double corner model.

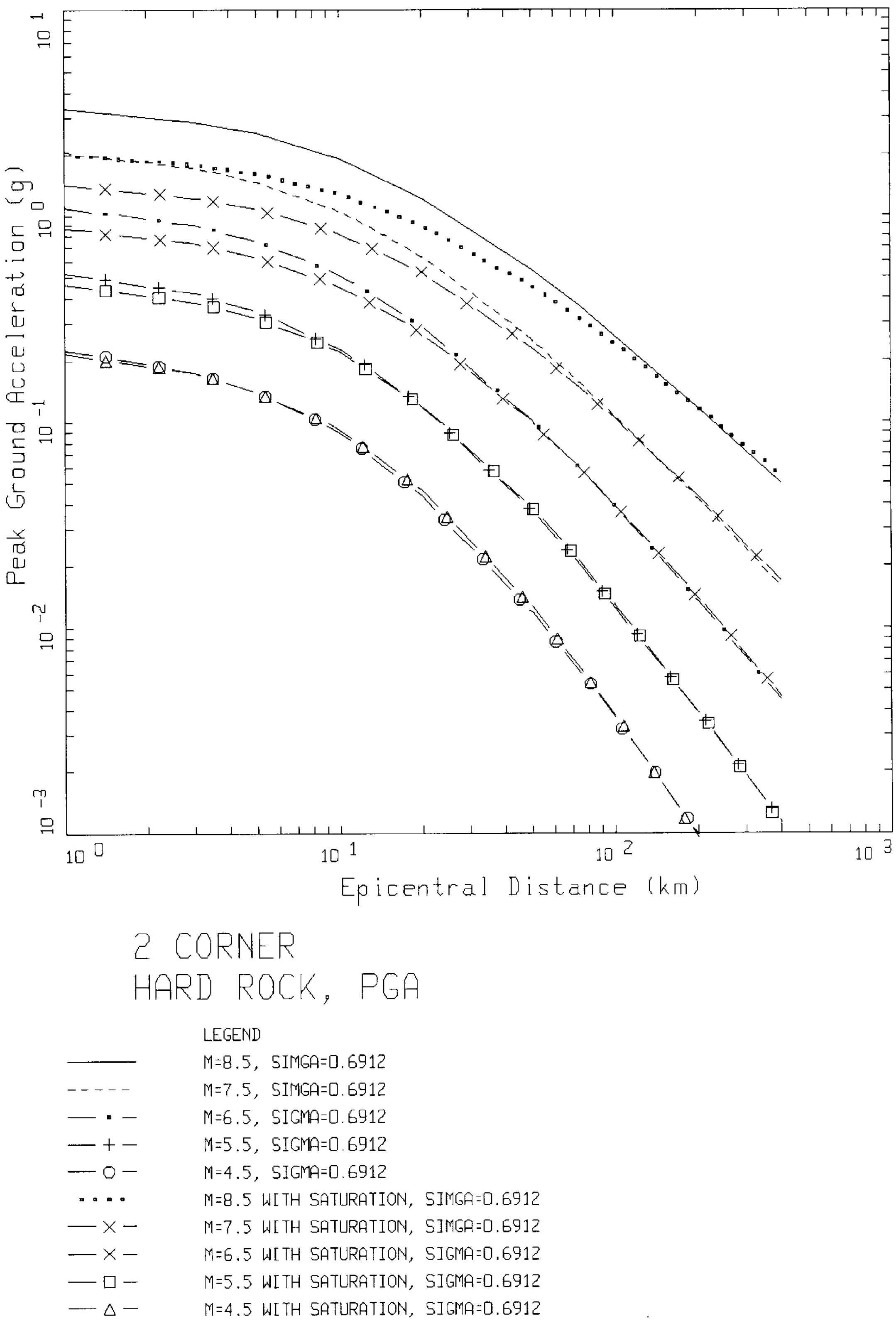

Figure 10. Attenuation of median peak horizontal accelerations at **M** 4.5, 5.5, 6.5, 7.5, and 8.5 for the double corner model, with and without saturation.

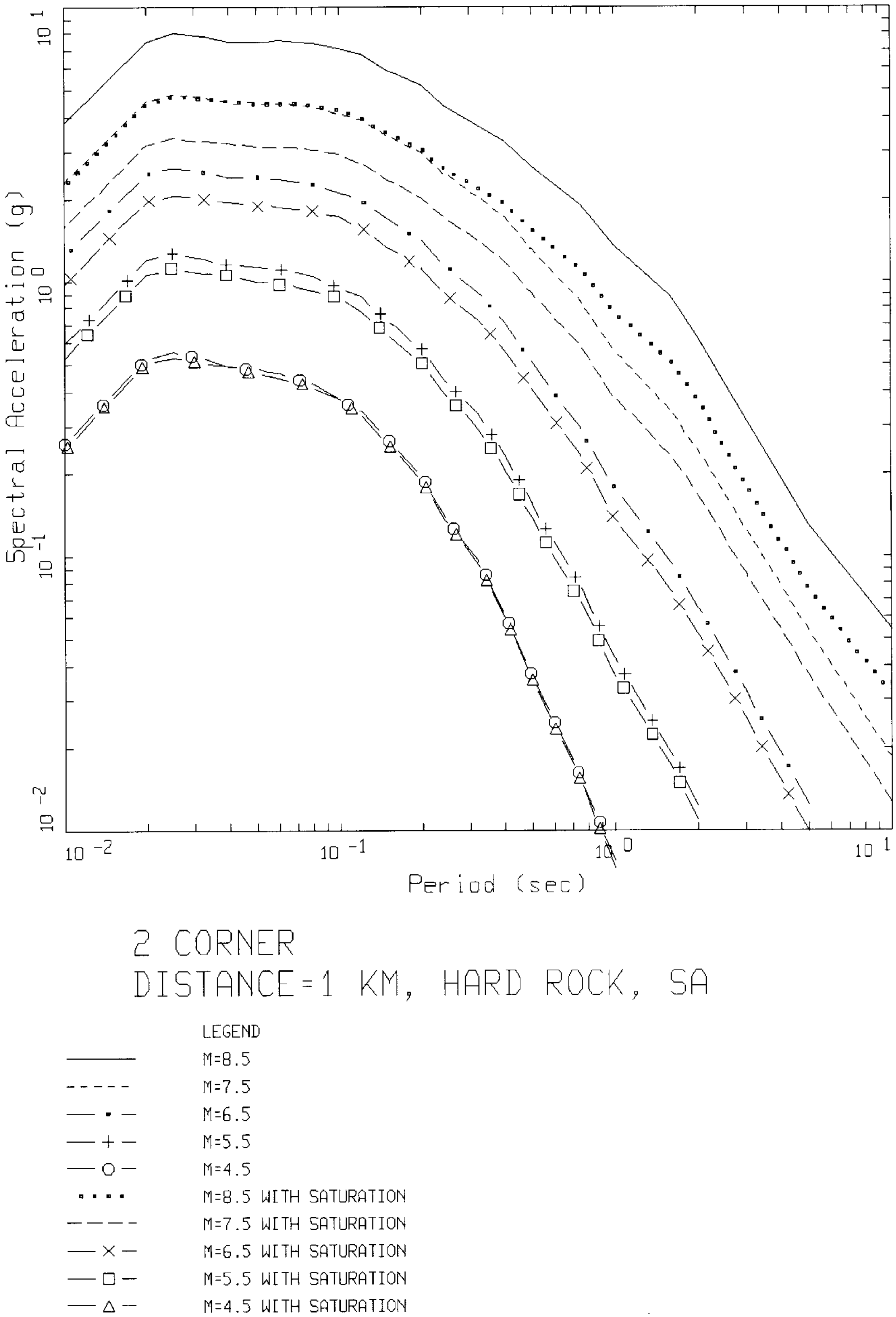

Figure 11. Median response spectra (5% damping) at a distance of 1 km for magnitudes **M** 4.5, 5.5, 6.5, 7.5, and 8.5 for the double corner model,

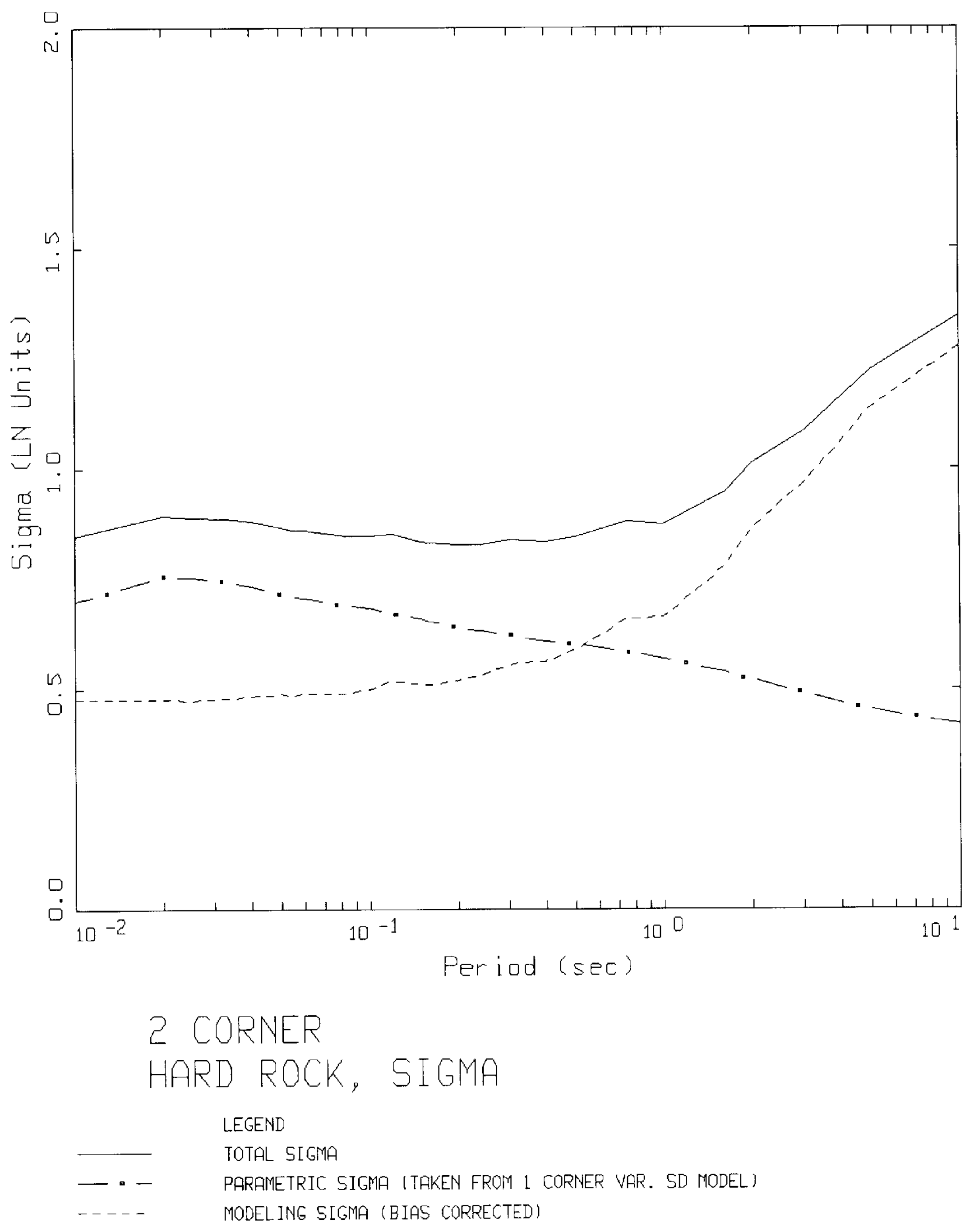

Figure 12. Estimates of total variability (uncertainty) for the attenuation model. Parametric variability is due to variation of variable stress drop, single corner frequency point-source parameters (Table 2) and fit of regression model (Table 6). Model variability is from validation exercises with 16 earthquakes (**M** 5.3 to 7.4) at 500 sites over the fault distance range of 1 to 460 km using the single corner frequency model (Appendix B).

APPENDIX A

STOCHASTIC GROUND MOTION MODEL DESCRIPTION

Background

In the context of strong ground motion, the term "stochastic" can be a fearful concept to some and may be interpreted to represent a fundamentally incorrect or inappropriate model (albeit the many examples demonstrating that it works well; Boore, 1983, 1986). To allay any initial misgivings, a brief discussion seems prudent to explain the term stochastic in the stochastic ground motion model.

The stochastic point-source model may be termed a spectral model in that it fundamentally describes the Fourier amplitude spectral density at the surface of a half-space (Hanks and McGuire, 1981). The model uses a Brune (1970, 1971) omega-square description of the earthquake source Fourier amplitude spectral density. This model is easily the most widely used and qualitatively validated source description available. Seismic sources ranging from **M** = -6 (hydrofracture) to **M** = 8 have been interpreted in terms of the Brune omega-square model in dozens of papers over the last 30 years. The general conclusion is that it provides a reasonable and consistent representation of crustal sources, particularly for tectonically active regions such as plate margins. A unique phase spectrum can be associated with the Brune source amplitude spectrum to produce a complex spectrum which can be propagated using either exact or approximate (1-2- or 3-D) wave propagation algorithms to produce single or multiple component time histories. In this context the model is not stochastic, it is decidedly deterministic and as exact and rigorous as one chooses. A two-dimensional array of such point-sources may be appropriately located on a fault surface (area) and fired with suitable delays to simulate rupture propagation on an extended rupture plane. As with the single point-source, any degree of rigor may be used in the wave propagation algorithm to produce multiple component or average horizontal component time histories. The result is a kinematic[1] finite-source model which has as its basis a source time history defined as a Brune pulse whose Fourier amplitude spectrum follows an omega-square model. This finite-fault model would be very similar to that used in published inversions for slip models if the 1-D propagation were treated using a reflectivity algorithm (Aki and Richards, 1980). This algorithm is a complete solution to the wave equation from static offsets (near-field terms) to an arbitrarily selected high frequency cutoff (generally 1-2 Hz).

Alternatively, to model the wave propagation more accurately, recordings of small earthquakes at the site of interest and with source locations distributed along the fault of interest may be used as empirical Green functions (Hartzell, 1978). To model the design earthquake, the empirical Green functions are delayed and summed in a manner to simulate rupture propagation (Hartzell, 1978). Provided a sufficient number of small

[1]Kinematic source model is one whose slip (displacement) is defined (imposed) while in a dynamic source model forces (stress) are defined (see Aki and Richards 1980 for a complete description).

earthquakes are recorded at the site of interest, the source locations adequately cover the expected rupture surface, and sufficient low frequency energy is present in the Green functions, this would be the most appropriate procedure to use if nonlinear site response is not an issue. With this approach the wave propagation is, in principle, exactly represented from each Green function source to the site. However, nonlinear site response is not treated unless Green function motions are recorded at a nearby rock outcrop with dynamic material properties similar to the rock underlying the soils at the site or recordings are made at depth within the site soil column. These motions may then be used as input to either total or effective stress site response codes to model nonlinear effects. Important issues associated with this approach include the availability of an appropriate nearby (1 to 2 km) rock outcrop and, for the downhole recordings, the necessity to remove all downgoing energy from the at-depth soil recordings. The downgoing energy must be removed from the downhole Green functions (recordings) prior to generating the control motions (summing) as only the upgoing wavefields are used as input to the nonlinear site response analyses. Removal of the downgoing energy from each recording requires multiple site response analyses which introduce uncertainty into the Green functions due to uncertainty in dynamic material properties and the numerical site response model used to separate the upgoing and downgoing wavefields.

To alleviate these difficulties one can use recordings well distributed in azimuth at close distances to a small earthquake and correct the recordings back to the source by removing wave propagation effects using a simple approximation (say 1/R plus a constant for crustal amplification and radiation pattern), to obtain an empirical source function. This source function can be used to replace the Brune pulse to introduce some natural (although source, path, and site specific) variation into the dislocation time history. If this is coupled to an approximate wave propagation algorithm (asymptotic ray theory) which includes the direct rays and those which have undergone a single reflection, the result is the empirical source function method (EPRI, 1993). Combining the reflectivity propagation (which is generally limited to frequencies ≤ 1-2 Hz due to computational demands) with the empirical source function approach (appropriate for frequencies ≥ 1 Hz; EPRI, 1993) results in a broad band simulation procedure which is strictly deterministic at low frequencies (where an analytical source function is used) and incorporates some natural variation at high frequencies through the use of an empirical source function (Sommerville et al., 1995).

All of these techniques are fundamentally similar, well founded in seismic source and wave propagation physics, and importantly, they are <u>all</u> approximate. Simply put, all models are wrong (approximate) and the single essential element in selecting a model is to incorporate the appropriate degree of rigor, commensurate with uncertainties and variabilities in crustal structure and site effects, through extensive validation exercises. It is generally felt that more complicated models produce more accurate results, however, the implications of more sophisticated models with the increased number of parameters which must be specified is often overlooked. This is not too serious a consequence in modeling past earthquakes since a reasonable range in parameter space can be explored to give the "best" results. However for future predictions, this increased rigor may carry undesirable baggage in increased parametric variability (Roblee et al., 1996). The effects of lack of knowledge (epistemic uncertainty; EPRI, 1993) regarding parameter values for future occurrences results in uncertainty or variability in ground motion predictions. It may easily be the case that a very simple model, such as the point-source model can have comparable, or even smaller, total variability (modeling plus parametric) than a much more rigorous model with an increased number of parameters (EPRI, 1993). What is desired in a model is sufficient sophistication such that it captures the dominant and stable features of source, distance, and site dependencies observed in strong ground motions. It is these considerations which led to the development of the stochastic point- and finite-source models and, in part, leads to the stochastic element of the models.

The stochastic nature of the point- and finite-source RVT models is simply the assumption made about the character of ground motion time histories that permits stable estimates of peak parameters (e.g. acceleration, velocity, strain, stress, oscillator response) to be made without computing detailed time histories (Hanks and McGuire, 1981; Boore, 1983). This process uses random vibration theory to relate a time domain peak value to the time history root-mean-square (RMS) value (Boore, 1983). The assumption of the character of the time history for this process to strictly apply is that it be normally distributed random noise and stationary (its statistics do not change with time) over its duration. A visual examination of any time history quickly reveals that this is clearly not the case: time histories (acceleration, velocity, stress, strain, oscillator) start, build up, and then diminish with time. However poor the assumption of stationary Gaussian noise may appear, the net result is that the assumption is weak enough to permit the approach to work surprisingly well, as numerous comparisons with recorded motions and both qualitative and quantative validations have shown (Hanks and McGuire, 1981; Boore, 1983, 1986; McGuire et al., 1984; Boore and Atkinson, 1987; Silva and Lee, 1987; Toro and McGuire, 1987; Silva et al., 1990; EPRI, 1993; Schneider et al., 1993; Silva and Darragh, 1995; Silva et al., 1997). Corrections to RVT are available to accommodate different distributions as well as non-stationarity and are usually applied to the estimation of peak oscillator response in the calculated response spectra (Boore and Joyner, 1984; Toro, 1985).

Point-source Model

The conventional stochastic ground motion model uses an ω-square source model (Brune, 1970, 1971) with a single corner frequency and a constant stress drop (Boore, 1983; Atkinson, 1984). Random vibration theory is used to relate RMS (root-mean-square) values to peak values of acceleration (Boore, 1983), and oscillator response (Boore and Joyner, 1984; Toro, 1985; Silva and Lee, 1987) computed from the power spectra to expected peak time domain values (Boore, 1983).

The shape of the acceleration spectral density, a(f), is given by

$$a(f) \;=\; C \, \frac{f^2}{1+\left(\dfrac{f}{f_0}\right)^2} \, \frac{MSUB0}{R} \, P(f)\, A(f)\; e^{-\frac{\pi f R}{\beta_0 Q(f)}} \qquad \text{(A-1)}$$

where

C $= (\frac{1}{\rho_0 \beta_0^3}) \bullet (2) \bullet (0.55) \bullet (\frac{1}{\sqrt{2}}) \bullet \pi.$

M_0 = seismic moment,
R = hypocentral distance,
β_0 = shear-wave velocity at the source,
ρ_0 = density at the source
Q(f) = frequency dependent quality factor (crustal damping),
A(f) = crustal amplification,
P(f) = high-frequency truncation filter,
f_o = source corner frequency.

C is a constant which contains source region density (ρ_0) and shear-wave velocity terms and accounts for the free-surface effect (factor of 2), the source radiation pattern averaged over a sphere (0.55) (Boore, 1986), and the partition of energy into two horizontal components ($1/\sqrt{2}$).

Source scaling is provided by specifying two independent parameters, the seismic moment (M_0) and the high-frequency stress parameter or stress drop ($\Delta\sigma$). The seismic moment is related to magnitude through the definition of moment magnitude **M** by the relation

$\log M_0 = 1.5\,\mathbf{M} + 16.05$ (Hanks and Kanamori, 1979) (A - 2).

The stress drop ($\Delta\sigma$) relates the corner frequency f_0 to M_0 through the relation

$f_0 = \beta_0\,(\Delta\sigma/8.44\,M_0)^{1/3}$ (Brune; 1970, 1971) (A - 3).

The stress drop is sometimes referred to as the high frequency stress parameter (Boore, 1983) (or simply the stress parameter) since it directly scales the Fourier amplitude spectrum for frequencies above the corner frequency (Silva, 1991; Silva and Darragh 1995). High (> 1 Hz) frequency model predictions are then very sensitive to this parameter (Silva, 1991; EPRI, 1993) and the interpretation of it being a stress drop or simply a scaling parameter depends upon how well real earthquake sources (on average) obey the omega-square scaling (Equation A-3) and how well they are fit by the single-corner-frequency model (Atkinson and Silva, 1997). If earthquakes truly have single-corner-frequency omega-square sources, the stress drop in Equation A-3 is a physical parameter and its values have a physical interpretation of the forces (stresses) accelerating the relative slip across the rupture surface. High stress drop sources are due to a smaller source (fault) area (for the same **M**) than low stress drop sources (Brune, 1970). Otherwise, it simply a high frequency ($f > f_o$) scaling or fitting parameter.

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The spectral shape of the single-corner-frequency ω-square source model is then described by the two free parameters M_0 and $\Delta\sigma$. The corner frequency increases with the shear-wave velocity and with increasing stress drop, both of which may be region dependent.

The crustal amplification accounts for the increase in wave amplitude as seismic energy travels through lower- velocity crustal materials from the source to the surface. The amplification depends on average crustal and near surface shear-wave velocity and density (Boore, 1986).

The P(f) filter is used in an attempt to model the observation that acceleration spectral density appears to fall off rapidly beyond some region- or site-dependent maximum frequency (Hanks, 1982; Silva and Darragh, 1995). This observed phenomenon truncates the high frequency portion of the spectrum and is responsible for the band-limited nature of the stochastic model. The band limits are the source corner frequency at low frequency and the high frequency spectral attenuation. This spectral fall-off at high frequency has been attributed to near-site attenuation (Hanks, 1982; Anderson and Hough, 1984) or to source processes (Papageorgiou and Aki, 1983) or perhaps to both effects. In the Anderson and Hough (1984) attenuation model, adopted here, the form of the P(f) filter is taken as

$$P(f, r) = e^{-\pi\kappa(r)f} \qquad \text{(A-4).}$$

Kappa (r) (κ(r) in Equation A-4) is a site and distance dependent parameter that represents the effect of intrinsic attenuation upon the wavefield as it propagates through the crust from source to receiver. Kappa (r) depends on epicentral distance (r) and on both the shear-wave velocity (β) and quality factor (Q_S) averaged over a depth of H beneath the site (Hough et al., 1988). At zero epicentral distance kappa (κ) is given by

$$\kappa(0) = \frac{H}{\bar{\beta}\,\bar{Q}_S} \qquad \text{(A-5),}$$

and is referred to as κ.

The bar in Equation A-5 represents an average of these quantities over a depth H. The value of kappa at zero epicentral distance is attributed to attenuation in the very shallow crust directly below the site (Hough and Anderson, 1988; Silva and Darragh, 1995). The intrinsic attenuation along this part of the path is not thought to be frequency dependent and is modeled as a frequency independent, but site and crustal region dependent, constant value of kappa (Hough et al., 1988; Rovelli et al., 1988). This zero epicentral distance kappa is the model implemented in this study.

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The crustal path attenuation from the source to just below the site is modeled with the frequency- dependent quality factor Q(f). Thus the distance component of the original κ(r) (Equation A-4) is accommodated by Q(f) and R in the last term of Equation A-1:

$$\kappa(r) = \frac{H}{\bar{\beta}\,\bar{Q}_S} + \frac{R}{\beta_0\, Q(f)} \tag{A-6}$$

The Fourier amplitude spectrum, a(f), given by Equation A-1 represents the stochastic ground motion model employing a Brune source spectrum that is characterized by a single corner frequency. It is a point source and models direct shear-waves in a homogeneous half-space (with effects of a velocity gradient captured by the A(f) filter, Equation A-1). For horizontal motions, vertically propagating shear-waves are assumed. Validations using incident inclined SH-waves accompanied with raytracing to find appropriate incidence angles leaving the source showed little reduction in uncertainty compared to results using vertically propagating shear-waves. For vertical motions, P/SV propagators are used coupled with raytracing to model incident inclined plane waves (EPRI, 1993). This approach has been validated with recordings from the 1989 **M** 6.9 Loma Prieta earthquake (EPRI, 1993).

Equation A-1 represents an elegant ground motion model that accommodates source and wave propagation physics as well as propagation path and site effects with an attractive simplicity. The model is appropriate for an engineering characterization of ground motion since it captures the general features of strong ground motion in terms of peak acceleration and spectral composition with a minimum of free parameters (Boore, 1983; McGuire et al., 1984; Boore, 1986; Silva and Green, 1988; Silva et al., 1988; Schneider et al., 1993; Silva and Darragh, 1995). An additional important aspect of the stochastic model employing a simple source description is that the region-dependent parameters may be evaluated by observations of small local or regional earthquakes. Region-specific seismic hazard evaluations can then be made for areas with sparse strong motion data with relatively simple spectral analyses of weak motion (Silva, 1992).

In order to compute peak time-domain values, i.e. peak acceleration and oscillator response, RVT is used to relate RMS computations to peak value estimates. Boore (1983) and Boore and Joyner (1984) present an excellent development of the RVT methodology as applied to the stochastic ground motion model. The procedure involves computing the RMS value by integrating the power spectrum from zero frequency to the Nyquist frequency and applying Parsevall's relation. Extreme value theory is then used to estimate the expected ratio of the peak value to the RMS value of a specified duration of the stochastic time history. The duration is taken as the inverse of the source corner frequency (Boore, 1983).

Factors that affect strong ground motions such as surface topography, finite and propagating seismic sources, laterally varying near-surface velocity and Q gradients, and random inhomogeneities along the propagation path are not included in the model. While some or all of these factors are generally present in any observation of ground motion and may exert controlling influences in some cases, the simple stochastic point-source model appears to be robust in predicting median or average properties of ground motion (Boore

1983, 1986; Schneider et al., 1993; Silva and Stark, 1993; Silva et al., 1997). The motivation for comprehensive validation exercises involving many earthquakes with a wide range in magnitudes, rupture distances, and site conditions is to capture unmodeled effects. The unmodeled effects which are random are captured in estimates of model uncertainty and those which are pervasive are captured in the estimates of model bias (see later sections). The combination of realistic, albeit simple, model physics with comprehensive validation exercises makes the stochastic point source ground motion model a powerful predictive and interpretative tool for engineering characterization of strong ground motion.

Finite-source Model Ground Motion Model

In the near-source region of large earthquakes, aspects of a finite-source including rupture propagation, directivity source-receiver geometry, and saturation of high-frequency (≥ 1 Hz) motions with increasing magnitude can be significant and may be incorporated into strong ground motion predictions. To accommodate these effects, a methodology that combines the aspects of finite-earthquake-source modeling techniques (Hartzell, 1978; Irikura 1983) with the stochastic point-source ground motion model has been developed to produce response spectra as well as time histories appropriate for engineering design (Silva et al., 1990; Silva and Stark, 1993; Schneider et al., 1993). The approach is very similar to the empirical Green function methodology introduced by Hartzell (1978) and Irikura (1983). In this case however, the stochastic point-source is substituted for the empirical Green function and peak amplitudes; PGA, PGV, and response spectra (when time histories are not produced) are estimated using random process theory.

Use of the stochastic point-source as a Green function is motivated by its demonstrated success in modeling ground motions in general and strong ground motions in particular (Boore, 1983, 1986; Silva and Stark, 1993; Schneider et al., 1993; Silva and Darragh, 1995) and the desire to have a model that is truly site- and region-specific. The model can accommodate a region specific Q(f), Green function sources of arbitrary moment or stress drop, and site specific kappa values and soil profiles. The necessity for having available regional and site specific recordings distributed over the rupture surface of a future earthquake or modifying possibly inappropriate empirical Green functions is eliminated.

For the finite-source characterization, a rectangular fault is discretized into NS subfaults of moment M_0^S. The empirical relationship

$$\log(A) = \mathbf{M} - 4.0, \quad A \text{ in km}^2 \tag{A-7}$$

is used to assign areas to both the target earthquake (if its rupture surface is not fixed) as well as to the subfaults. This relation results from regressing log area on **M** using the data of Wells and Coppersmith (1994). In the regression, the coefficient on **M** is set to unity which implies a constant static stress drop of about 30 bars (Equation A-9). This is consistent with the general observation of a constant static stress drop for earthquakes based on aftershock locations (Wells and Coppersmith 1994). The static stress drop, defined by Equation A-10, is related to the average slip over the rupture surface as well as rupture area. It is theoretically identical to the stress drop in Equation A-3 which defines the omega-square source corner frequency assuming the rupture surface is a circular crack model (Brune, 1970; 1971). The stress drop determined by the source corner frequency (or source duration) is usually estimated through the Fourier amplitude spectral density while the static stress drop uses the moment magnitude and an estimate of the rupture area. The two estimates for the same earthquake seldom yield the same values with the static generally being the smaller. In a recent study (Silva et al., 1997), the average stress drop based on Fourier amplitude spectra determined from an empirical attenuation relation (Abrahamson and Silva, 1997) is about 70 bars while the average static stress drop for the crustal earthquakes studied by Wells and Coppersmith (1994) is about 30 bars. These results reflect a general factor of about 2 on average between the two values. These large differences may simply be the result of using an inappropriate estimate of rupture area as the zone of actual slip is difficult to determine unambiguously. In general however, even for individual earthquakes, the two stress drops scale similarly with high static stress drops (> 30 bars) resulting in large high frequency (> 1 Hz for **M** ≥ 5) ground motions which translates to high corner frequencies (Equation A-3).

The subevent magnitude M_S is generally taken in the range of 5.0-6.5 depending upon the size of the target event. M_S 5.0 is used for crustal earthquakes with **M** in the range of 5.5 to 8.0 and M_S 6.4 is used for large subduction earthquakes with **M** > 7.5. The value of NS is determined as the ratio of the target event area to the subfault area. To constrain the proper moment, the total number of events summed (N) is given by the ratio of the target event moment to the subevent moment. The subevent and target event rise times (duration of slip at a point) are determined by the equation

$$\log \tau = 0.33 \log M_0 - 8.54 \tag{A-8}$$

which results from a fit to the rise times used in the finite-fault modeling exercises, (Silva et al., 1997). Slip on each subfault is assumed to continue for a time τ. The ratio of target-to-subevent rise times is given by

$$\frac{\tau}{\tau^s} = 10^{0.5\,(\mathrm{M}\ -\ \mathrm{MSUPs})} \tag{A-9}$$

and determines the number of subevents to sum in each subfault. This approach is generally referred to as the constant-rise-time model and results in variable slip velocity for nonuniform slip distributions. Alternatively, one can assume a constant slip velocity (as do Beresnev and Atkinson, 2002) resulting in a variable-rise-time model for heterogenous slip distributions. This approach was implemented and validations resulted in an overall “best” average slip velocity of about 70 cm/sec, with no significant improvement over a magnitude dependent rise time (Equation A-8). The feature is retained as an option in the simulation code.

Recent modeling of the Landers (Wald and Heaton, 1994), Kobe (Wald, 1996) and Northridge (Hartzell et al. 1996) earthquakes suggests that a mixture of both constant rise time and constant slip velocity may be present. Longer rise times seem to be associated with areas of larger slip with the ratio of slip-to-rise time (slip velocity) being depth dependent. Lower slip velocities (longer rise times) are associated with shallow slip resulting in relatively less short period seismic radiation. This result may explain the general observation that shallow slip is largely aseismic. The significant contributions to strong ground motions appear to originate at depths exceeding about 4 km (Campbell, 1993; Boore et al., 1994) as the fictitious depth term in empirical attenuation relation (Abrahamson and Silva, 1997; Boore et al., 1997). Finite-fault models generally predict unrealistically large strong ground motions for large shallow (near surface) slip using rise times or slip velocities associated with deeper (> 4 km) zones of slip. This is an important and unresolved issue in finite-fault modeling and the general approach is constrain the slip to relatively small values in the top 2 to 4 km. For the composite source model, the approach is to taper the subevent stress drop to zero at the ground surface (Yehua Zeng, personal communication 1999). A more thorough analysis is necessary, ideally using several well validated models, before this issue can be satisfactorily resolved.

To introduce heterogeneity of the earthquake source process into the stochastic finite-fault model, the location of the sub-events within each subfault (Hartzell, 1978) are randomized as well as the subevent rise time ($\sigma_{\ln} = 0.8$). The stress drop of the stochastic point-source Green function is taken as 30 bars, consistent with the static value based on the **M** 5.0 subevent area using the equation

$$\Delta\sigma = \frac{7}{16}\left(\frac{M_e}{R_e^3}\right) \qquad \text{(Brune, 1970, 1971)} \qquad \text{(A-10)}$$

where R_e is the equivalent circular radius of the rectangular sub-event.

Different values of slip are assigned to each subfault as relative weights so that asperities or non-uniform slip can be incorporated into the methodology. For validation exercises, slip models are taken from the literature and are based on inversions of strong motion as well as regional or teleseismic recordings. To produce slip distributions for future earthquakes, random slip models are generated based on a statistical asperity model with parameters calibrated to the published slip distributions. This approach has been validated by comparing the modeling uncertainty and bias estimates for the Loma Prieta and Whittier Narrows earthquakes using motion at each site averaged over several (30) random slip models to the bias and uncertainty estimates using the published slip model. The results show nearly identical bias and uncertainty estimates suggesting that averaging the motions over random slip models produces as accurate a prediction at a site as a single motion computed using the "true" slip model which is determined from inverting actual recordings.

The rupture velocity is taken as depth independent at a value of 0.8 times the shear-wave velocity, generally at the depth of the dominant slip. This value is based on a number of studies of source rupture processes which also suggest that rupture velocity is non-uniform. To capture the effects of non-uniform rupture velocity, a random component is added through the randomized location of the subevents within each subfault. The radiation pattern is computed for each subfault, a random component added, and the RMS applied to the motions computed at the site when modeling an average horizontal component. To model individual horizontal components, the radiation pattern for each subfault is used to scale each subfaults contribution to the final summed motion.

The ground-motion time history at the receiver is computed by summing the contributions from each subfault associated with the closest Green function, transforming to the frequency domain, and convolving with the appropriate Green function spectrum (Equation A-1). The locations of the Green functions are generally taken at center of each subfault for small subfaults or at a maximum separation of about 5 to 10 km for large subfaults. As a final step, the individual contributions associated with each Green function are summed in the frequency domain, multiplied by the RMS radiation pattern, and the resultant power spectrum at the site is computed. The appropriate duration used in the RVT computations for PGA, PGV, and oscillator response is computed by transforming the summed Fourier spectrum into the time domain and computing the 5 to 75% Arias intensity (Ou and Herrmann, 1990).

As with the point-source model, crustal response effects are accommodated through the amplification factor (A(f)) or by using vertically propagating shear waves through a vertically heterogenous crustal structure. Propagation path damping, through the Q(f) model, is incorporated from each fault element to the site. Near-surface crustal damping is incorporated through the kappa operator (Equation A-1). To model crustal propagation path effects, the raytracing method of Ou and Herrmann (1990) is applied from each subfault to the site.

Time histories may be computed in the process as well by simply adding a phase spectrum appropriate to the subevent earthquake. The phase spectrum can be extracted from a recording made at close distance to an earthquake of a size comparable to that of the subevent (generally **M** 5.0 to 6.5). Interestingly, the phase spectrum need not be from a recording in the region of interest (Silva et al., 1989). A recording in WNA (Western North America) can effectively be used to simulate motions appropriate to ENA (Eastern North America). Transforming the Fourier spectrum computed at the site into the time domain results in a computed time history which then includes all of the aspects of rupture propagation and source finiteness, as well as region specific propagation path and site effects.

For fixed fault size, mechanism, and moment, the specific source parameters for the finite-fault are slip distribution, location of nucleation point, and site azimuth. The propagation path and site parameters remain identical for both the point- and finite-source models.

Partition and assessment of ground motion variability

APPENDIX A

An essential requirement of any numerical modeling approach, particularly one which is implemented in the process of defining design ground motions, is a quantative assessment of prediction accuracy. A desirable approach to achieving this goal is in a manner which lends itself to characterizing the variability associated with model predictions. For a ground motion model, prediction variability is comprised of two components: modeling variability and parametric variability. Modeling variability is a measure of how well the model works (how accurately it predicts ground motions) when specific parameter values are known. Modeling variability is measured by misfits of model predictions to recorded motions through validation exercises and is due to unaccounted for components in the source, path, and site models (i.e. a point-source cannot model the effects of directivity and linear site response cannot accommodate nonlinear effects). Results from a viable range of values for model parameters (i.e., slip distribution, soil profile, G/G_{max} and hysteretic damping curves, etc). Parametric variability is the sensitivity of a model to a viable range of values for model parameters. The total variability, modeling plus parametric, represents the variance associated with the ground motion prediction and, because it is a necessary component in estimating fractile levels, may be regarded as important as median predictions.

Both the modeling and parametric variabilities may have components of randomness and uncertainty. Table A.1 summarizes the four components of total variability in the context of ground motion predictions. Uncertainty is that portion of both modeling and parametric variability which, in principle, can be reduced as additional information becomes available, whereas randomness represents the intrinsic or irreducible component of variability for a given model or parameter. Randomness is that component of variability which is intrinsic or irreducible <u>for a given model.</u> The uncertainty component reflects a lack of knowledge and may be reduced as more data are analyzed. For example, in the point-source model, stress drop is generally taken to be independent of source mechanism as well as tectonic region and is found to have a standard error of about 0.7 (natural log) for the CEUS (EPRI, 1993). This variation or uncertainty plus randomness in $\Delta\sigma$ results in a variability in ground motion predictions for future earthquakes. If, for example, it is found that normal faulting earthquakes have generally lower stress drops than strike-slip which are, in turn, lower than reverse mechanism earthquakes, perhaps much of the variability in $\Delta\sigma$ may be reduced. In extensional regimes, where normal faulting earthquakes are most likely to occur, this new information may provide a reduction in variability (uncertainty component) for stress drop, say to 0.3 or 0.4 resulting in less ground motion variation due to a lack of knowledge of the mean or median stress drop. There is, however, a component of this stress drop variability which can <u>never</u> be reduced in the context of the Brune model. This is simply due to the heterogeneity of the earthquake dynamics which is not accounted for in the model and results in the randomness component of parametric variability in stress drop. A more sophisticated model may be able to accommodate or model more accurately source dynamics but, perhaps, at the expense of a larger number of parameters and increased parametric uncertainty (i.e. the finite-fault with slip model and nucleation point as unknown parameters for future earthquakes). That is, more complex models typically seek to reduce modeling randomness by more closely modeling physical phenomena. However, such models often require more comprehensive sets of observed data to constrain additional model parameters, which generally leads to increased parametric variability. If the increased parametric variability is primarily in the form of uncertainty, it is possible to reduce total variability, but only at the additional expense of constraining the additional parameters. Therefore, existing knowledge and/or available resources may limit the ability of more complex models to reduce total variability.

The distinction of randomness and uncertainty is model driven and somewhat arbitrary. The allocation is only important in the context of probabilistic seismic hazard analyses as uncertainty is treated as alternative hypotheses in logic trees while randomness is integrated over in the hazard calculation (Cornell, 1968). For example, the uncertainty component in stress drop may be treated by using an N-point approximation to the stress drop distribution and assigning a branch in a logic tree for each stress drop and associated weight. A reasonable three point approximation to a normal distribution is given by weights of 0.2, 0.6, 0.2 for expected 5%, mean, and 95% values of stress drop respectively. If the distribution of uncertainty in stress drop was such that the 5%, mean, and 95% values were 50, 100, and 200 bars respectively, the stress drop branch on a logic tree would have 50, and 200 bars with weights of 0.2 and 100 bars with a weight of 0.6. The randomness component in stress drop variability would then be formally integrated over in the hazard calculation.

Assessment of Modeling Variability

Modeling variability (uncertainty plus randomness) is usually evaluated by comparing response spectra computed from recordings to predicted spectra and is a direct assessment of model accuracy. The modeling variability is defined as the standard error of the residuals of the log of the average horizontal component (or vertical component) response spectra. The residual is defined as the difference of the logarithms of the observed average 5% damped acceleration response spectra and the predicted response spectra. At each period, the residuals are squared, and summed over the total number of sites for one or all earthquakes modeled. Dividing the resultant sum by the number of sites results in an estimate of the model variance. Any model bias (average offset) that exists may be estimated in the process (Abrahamson et al., 1990; EPRI, 1993) and used to correct (lower) the variance (and to adjust the median as well). In this approach, the modeling variability can be separated into randomness and uncertainty where the bias corrected variability represents randomness and the total variability represents randomness plus uncertainty. The uncertainty is captured in the model bias as this may be reduced in the future by refining the model. The remaining variability (randomness) remains irreducible for <u>this</u> model. In computing the variance and bias estimates only the frequency range between processing filters at each site (minimum of the 2 components) should be used.

Assessment of Parametric Variability

Parametric variability, or the variation in ground motion predictions due to uncertainty and randomness in model parameters is difficult to assess. Formally, it is straight-forward in that a Monte Carlo approach may be used with each parameter randomly sampled about its mean (median) value either individually for sensitivity analyses (Silva, 1992; Roblee et al., 1996) or in combination to estimate the total parametric variability (Silva, 1992; EPRI, 1993). In reality, however, there are two complicating factors.

The first factor involves the specific parameters kept fixed with all earthquakes, paths, and sites when computing the modeling variability. These parameters are then implicitly included in modeling variability provided the data sample a sufficiently wide range in source, path, and site conditions. The parameters which are varied during the assessment of modeling variation should have a degree of uncertainty and randomness associated with them for the next earthquake. Any ground motion prediction should then have a variation reflecting this lack of knowledge and randomness in the free parameters.

An important adjunct to fixed and free parameters is the issue of parameters which may vary but by fixed rules. For example, source rise time (Equation A-8) is magnitude dependent and in the stochastic finite-source model is specified by an empirical relation. In evaluating the modeling variability with different magnitude earthquakes, rise time is varied, but because it follows a strict rule, any variability associated with rise time variation is counted in modeling variability. This is strictly true only if the sample of earthquakes has adequately spanned the space of magnitude, source mechanism, and other factors which may affect rise time. Also, the earthquake to be modeled must be within that validation space. As a result, the validation or assessment of model variation should be done on as large a number of earthquakes of varying sizes and mechanisms as possible.

The second, more obvious factor in assessing parametric variability is a knowledge of the appropriate distributions for the parameters (assuming correct values for median or mean estimates are known). In general, for the stochastic models, median parameter values and uncertainties are based, to the extent possible, on evaluating the parameters derived from previous earthquakes (Silva, 1992; EPRI, 1993).

The parametric variability is site, path, and source dependent and must be evaluated for each modeling application (Roblee et al., 1996). For example, at large source-to-site distances, crustal path damping may control short-period motions. At close distances to a large fault, both the site and finite-source (asperity location and nucleation point) may dominate, and, depending upon site characteristics, the source or site may control different frequency ranges (Silva, 1992; Roblee et al., 1996). Additionally, level of control motion may affect the relative importance of G/G_{max} and hysteretic damping curves.

In combining modeling and parametric variations, independence is assumed (covariance is zero) and the variances are simply added to give the total variability.

$$_{\ln}\sigma^2_T = {}_{\ln}\sigma^2_{\mathbf{M}} + {}_{\ln}\sigma^2_P \quad (A\text{-}11),$$

[2]

[2]Strong ground motions are generally considered to be log normally distributed.

where

$_{\ln}\sigma^2_{\mathbf{M}}$ = modeling variation,

$_{\ln}\sigma^2_{P}$ = parametric variation.

Validation Of The Point- and Finite-Source Models

In a recent Department of Energy sponsored project (Silva et al., 1997), both the point- and finite-source stochastic models were validated in a systematic and comprehensive manner. In this project, 16 well recorded earthquakes were modeled at about 500 sites. Magnitudes ranged from **M** 5.3 to **M** 7.4 with fault distances from about 1 km out to 218 km for WUS earthquakes and 460 km for CEUS earthquakes. This range in magnitude and distance as well as number of earthquakes and sites results in the most comprehensively validated model currently available to simulate strong ground motions.

For these exercises, regional Q(f) models and point source stress drops were determined through inversions using the strong motion recordings (Silva et al., 1997). Small strain WUS rock and soil kappa values were set to 0.04 sec, the average from the inversions of small strain data. CEUS rock site kappa values were fixed at inversion values, which averaged about 0.02 sec and ranged from 0.004 to 0.06 sec. For the finite source parameters, slip models and nucleation points were taken from the literature (Silva et al., 1997). Point-source depths were taken as the depth of the center of the largest asperity in the slip models while point-source distance used the closest distance to the surface projection of the rupture surface.

A unique aspect of this validation is that rock and soil sites were modeled using generic rock and soil profiles and equivalent-linear site response. Validations done with other simulation procedures typically neglect site conditions as well as nonlinearity resulting in ambiguity in interpretation of the simulated motions.

Point-Source Model

Final model bias and variability estimates for the point-source model are shown in Figure A1. Over all the sites (Figure A1) the bias is slightly positive for frequencies greater than about 10 Hz and is near zero from about 10 Hz to 1 Hz. Below 1 Hz, a stable point-source overprediction is reflected in the negative bias. The analyses are considered reliable down to about 0.3 Hz (3.3 sec) where the point-source shows about a 40% overprediction.

The model variability is low, about 0.5 above about 3 to 4 Hz and increases with decreasing frequency to near 1 at 0.3 Hz. Above 1 Hz, there is little difference between the total variability (uncertainty plus randomness) and randomness (bias corrected variability) reflecting the near zero bias estimates. Below 1 Hz there is considerable uncertainty contributing to the total variability suggesting that the model can be measurably improved as its predictions tend to be consistently high at very low frequencies ($\leq$ 1 Hz). This stable misfit may be interpreted as the presence of a second corner frequency for WNA sources (Atkinson and Silva, 1997).

Finite-Source Model

For the finite-fault, Figure A2 shows the corresponding bias and variability estimates. For all the sites, the finite-source model provides slightly smaller bias estimates and, surprisingly, slightly higher variability for frequencies exceeding about 5 Hz. The low frequency ($\leq$ 1 Hz) point-source overprediction is not present in the finite-source results, indicating that it is giving more accurate predictions than the point-source model over a broad frequency range, from about 0.3 Hz (the lowest frequency of reliable analyses) to the highest frequency of the analyses.

In general, for frequencies of about 1 Hz and above the point-source and finite-source give comparable results: the bias estimates are small (near zero) and the variabilities range from about 0.5 to 0.6. These estimates are low considering the analyses are based on a data set comprised of earthquakes with **M** less than **M** 6.5 (288 of 513 sites) and high frequency ground motion variance decreases with increasing magnitude, particularly above **M** 6.5 (Youngs et al., 1995) Additionally, for the vast majority of sites, generic site conditions were used (inversion kappa values were used for only the Saguenay and Nahanni earthquake analyses, 25 rock sites). As a result, the model variability (mean = 0) contains the total uncertainty and randomness contribution for the site. The parametric variability due to uncertainty and randomness in site parameters: shear-wave velocity, profile depth, G/G_{max} and hysteretic damping curves need not be added to the model variability estimates. It is useful to perform parametric variations to assess site parameter sensitivities on the ground motions, but only source and path damping Q(f) parametric variabilities require assessment on a site specific basis and added to the model variability. The source uncertainty and randomness components include point-source stress drop as well as source depth and finite-source slip model and nucleation point (Silva, 1992).

The general approach taken in these validations is to have few free parameters and accept a relatively large model misfit. This approach relaxes the need to develop appropriate distributions for poorly resolved parameters such as spatially varying rise times and rupture velocity as well as non-planar rupture surfaces (e.g. Landers, Kobe, and Kocaeli earthquakes). An alternative approach is to adjust these suites of parameters, which naturally improves the fits to recorded motions and results in smaller modeling uncertainties. However, unless independent information is available to constrain these parameters for future earthquakes, they <u>must</u> be <u>appropriately</u> counted as parametric variability. This may result in the total variability remaining comparable between the two approaches. This concept parallels the utility of increased model complexity, i.e., simple verses complex models. More complex models may increase an understanding of physical processes but, in the context of predicting motions due to the <u>next</u> earthquake, increased model complexity may not provide more accurate estimates of strong ground motions, again unless independent information is available to constrain potential ranges in some or all of the free parameters.

A summary of fixed and free parameters for the implementation of the stochastic point and finite source models presented here is listed in Table 2.

Empirical Attenuation Model

As an additional assessment of the stochastic models, bias and variability estimates were made over the same earthquakes (except Saguenay since it was not used in the regressions) and sites using a recently develop empirical attenuation relation (Abrahamson and Silva, 1997). For all the sites, the estimates are shown in Figure A3. Interestingly, the point-source overprediction below about 1 Hz is present in the empirical relation perhaps suggesting that this suite of earthquakes possess lower than expected motions in this frequency range as the empirical model does not show this bias over all earthquakes ($\approx$ 50) used in its development. Comparing these results to the point- and finite-source results (Figures A1 and A2) show comparable bias and variability estimates. For future predictions, source and path damping parametric variability must be added to the numerical simulations which will contribute a σ_{ln} of about 0.2 to 0.4, depending upon frequency, source and path conditions, and site location. This will raise the modeling variability from about 0.50 to the range of 0.54 to 0.64, about 10 to 30%. These values are still comparable to the variability of the empirical relation indicating that the point- and finite-source numerical models perform about as well as a recently developed empirical attenuation relation for the validation earthquakes and sites.

These results are very encouraging and provide an additional qualitative validation of the point- and finite-source models. Paranthetically this approach provides a rational basis for evaluating empirical attenuation models.

APPENDIX A

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Table A.1

CONTRIBUTIONS TO TOTAL VARIABILITY
IN GROUND MOTION MODELS

	Modeling Variability	Parametric Variability
Uncertainty *(also Epistemic Uncertainty)*	Modeling Uncertainty: Variability in predicted motions resulting from particular model assumptions, simplifications and/or fixed parameter values. *Can be reduced by adjusting or "calibrating" model to better fit observed earthquake response.*	Parametric Uncertainty: Variability in predicted motions resulting from incomplete data needed to characterize parameters. *Can be reduced by collection of additional information which better constrains parameters*
Randomness *(also Aleatory Uncertainty)*	Modeling Randomness: Variability in predicted motions resulting from discrepancies between model and actual complex physical processes. *Cannot be reduced for a given model form.*	Parametric Randomness: Variability in predicted motions resulting from inherent randomness of parameter values. *Cannot be reduced a priori[***] by collection of additional information.*

[***]Some parameters (e.g. source characteristics) may be well defined after an earthquakes.

Table A.2

FIXED AND FREE PARAMETERS

Fixed Parameters

Regional Curstal Model
Rock and Soil Generic Profiles
Kappa
G/Gmax and Hysteric Damping Curves
Finite Source Rise Time
Finite Source Rupture Velocity

Free Parameters

Regional Q(f) Model
Point Source Stress Drop and Depth
Finite Source Slip Model and Nucleation Point

APPENDIX A

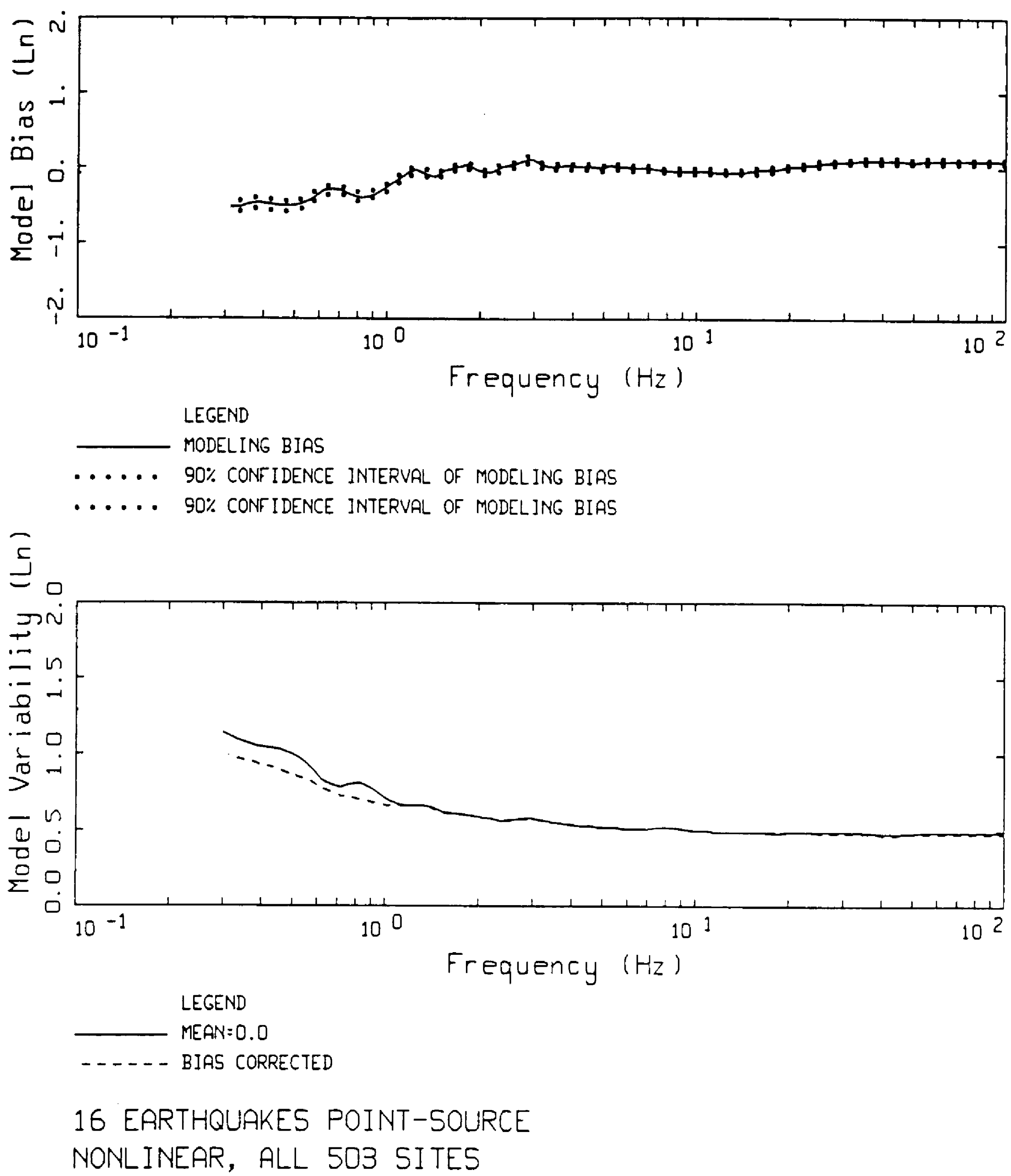

Figure A1. Model bias and variability estimates for all earthquakes computed over all 503 sites for the point-source model.

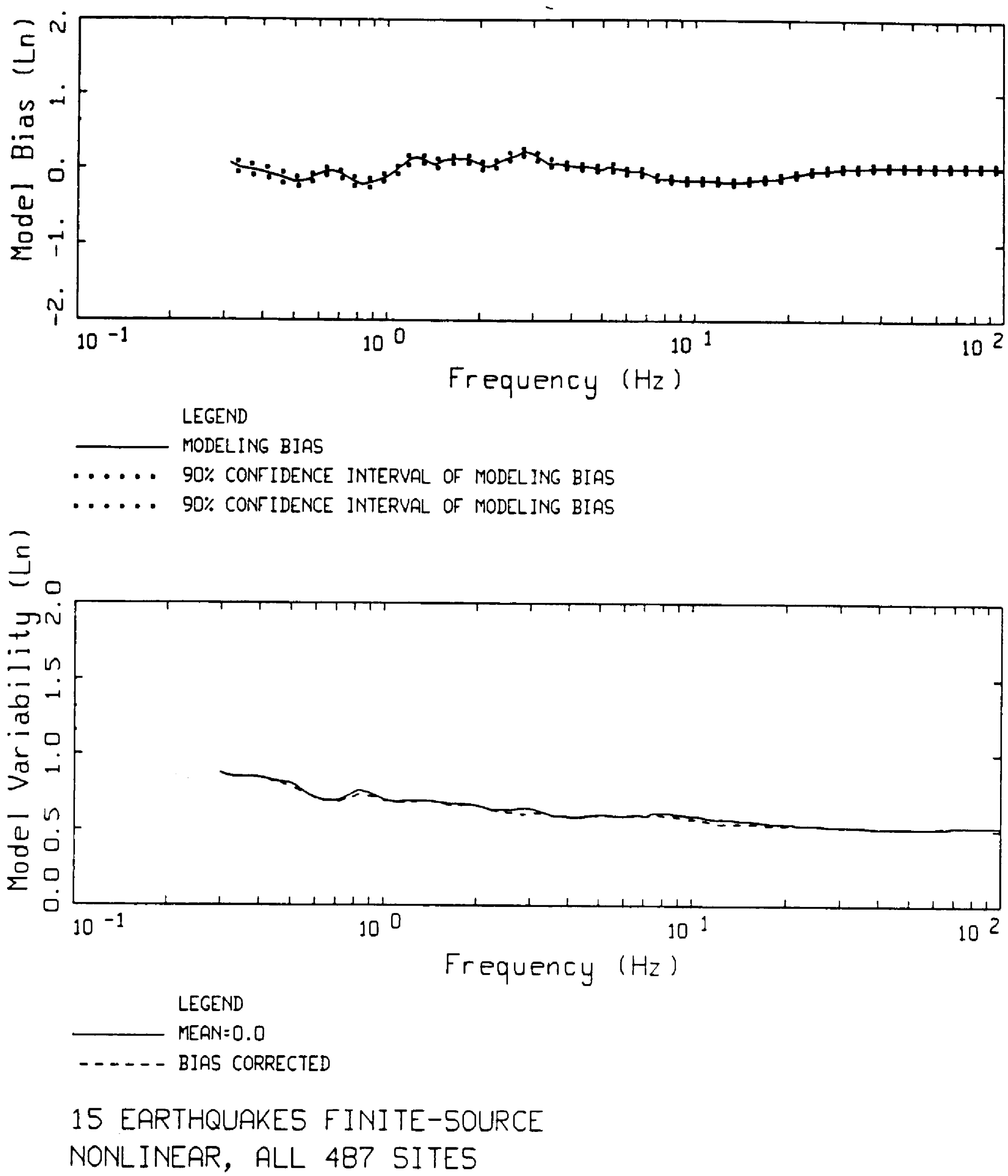

Figure A2. Model bias and variability estimates for all earthquakes computed over all 487 sites for the finite-source model.

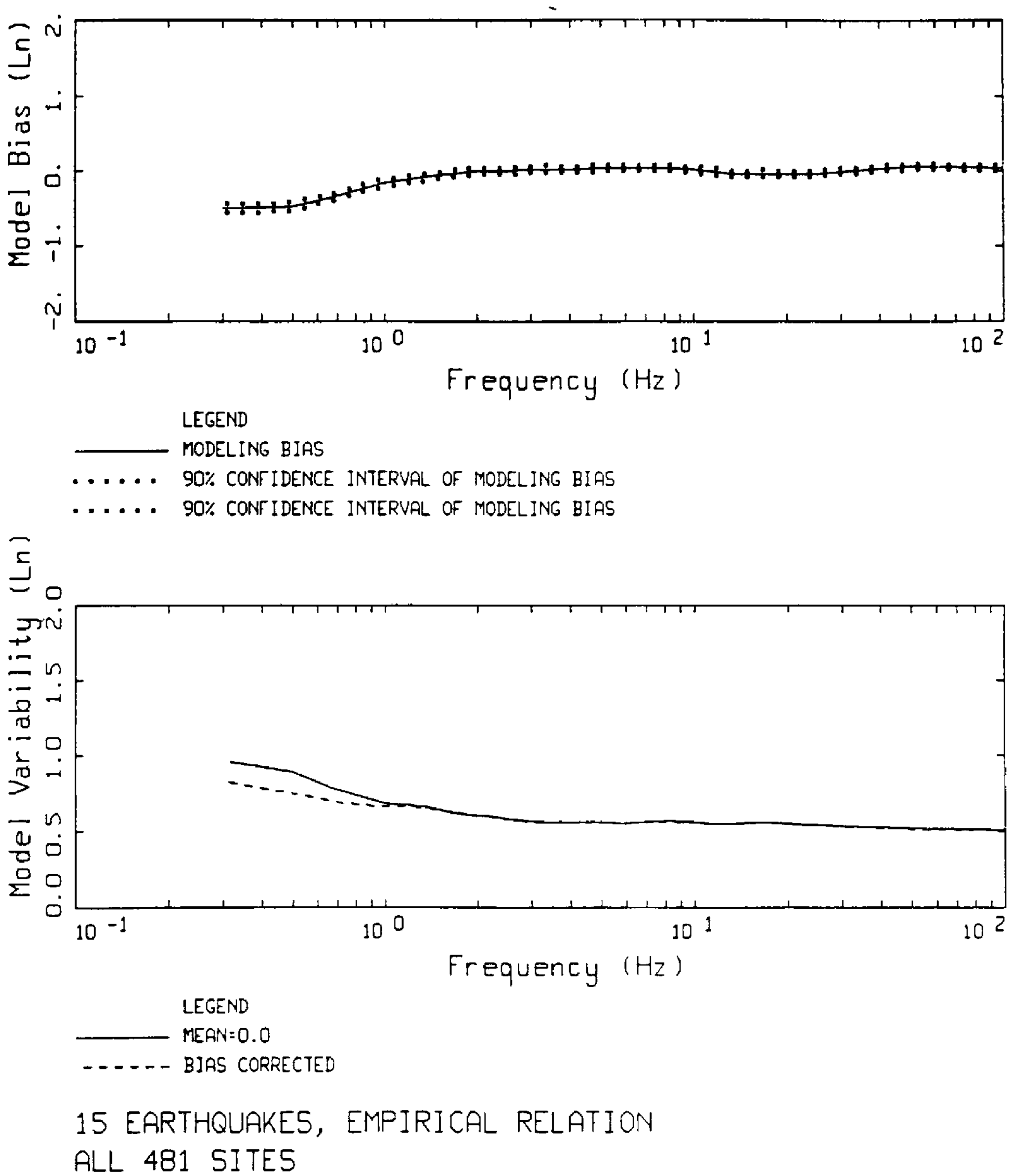

Figure A3. Model bias and variability estimates for all earthquakes computed over all 481 sites for the empirical model.

APPENDIX B

SITE RESPONSE ANALYSIS METHOD

Development of Site Specific Soil Motions

The conventional approach to estimating the effects of site-specific site conditions on strong ground motions involves development of a set (1, 2, or 3 component) of time histories compatible with the specified outcrop response spectra to serve as control (or input) motions. The control motions are then used to drive a nonlinear computational formulation to transmit the motions through the profile. Simplified analyses generally assume vertically propagating shear-waves for horizontal components and vertically propagating compression-waves for vertical motions. These are termed one-dimensional site response analyses.

Equivalent-Linear Computational Scheme

The computational scheme which has been most widely employed to evaluate one-dimensional site response assumes vertically-propagating plane shear-waves. Departures of soil response from a linear constitutive relation are treated in an approximate manner through the use of the equivalent-linear approach.

The equivalent-linear approach, in its present form, was introduced by Seed and Idriss (1970). This scheme is a particular application of the general equivalent-linear theory developed by Iwan (1967). Basically, the approach is to approximate a second order nonlinear equation, over a limited range of its variables, by a linear equation. Formally this is done in such a way that the average of the difference between the two systems is minimized. This was done in an ad-hoc manner for ground response modeling by defining an effective strain which is assumed to exist for the duration of the excitation. This value is usually taken as 65% of the peak time-domain strain calculated at the midpoint of each layer, using a linear analysis. Modulus reduction and hysteretic damping curves are then used to define new parameters for each layer based on the effective strain computations. The linear response calculation is repeated, new effective strains evaluated, and iterations performed until the changes in parameters are below some tolerance level. Generally a few iterations are sufficient to achieve a strain-compatible linear solution.

This stepwise analysis procedure was formalized into a one-dimensional, vertically propagating shear-wave code called SHAKE (Schnabel et al., 1972). Subsequently, this code has easily become the most widely used analysis package for one-dimensional site response calculations.

The advantages of the equivalent-linear approach are that parameterization of complex nonlinear soil models is avoided and the mathematical simplicity of a linear analysis is preserved. A truly nonlinear approach requires the specification of the shapes of hysteresis curves and their cyclic dependencies through an increased number of material parameters. In the equivalent-linear methodology the soil data are utilized directly and, because at each iteration the problem is linear and the material properties are frequency independent, the damping is rate independent and hysteresis loops close.

Careful validation exercises between equivalent-linear and fully nonlinear formulations using recorded motions from 0.05 to 0.50g showed little difference in results (EPRI, 1993). Both formulations compared very favorably to recorded motions suggesting both the adequacy of the vertically propagating shear-wave model and the approximate equivalent-linear formulation. While the assumptions of vertically propagating shear-waves and equivalent-linear soil response certainly represent approximations to actual conditions, their combination has achieved demonstrated success in modeling observations of site effects and represent a stable, mature, and reliable means of estimating the effects of site conditions on strong ground motions (Schnabel et al., 1972; Silva et al., 1988; Schneider et al., 1993; EPRI, 1993).

To accommodate both uncertainty and randomness in dynamic material properties, analyses are typically done for the best estimate shear-wave velocity profile as well as upper- and lower-range profiles. The upper- and lower-ranges are usually specified as twice and one-half the best estimate shear-wave moduli. Depending upon the nature of the structure, the final design spectrum is then based upon an envelope or average of the three spectra.

For vertical motions, the SHAKE code is also used with compression-wave velocities and damping substituted for the shear-wave values. To accommodate possible nonlinear response on the vertical component, since modulus reduction and hysteretic damping curves are not generally available for the constrained modulus, the low-strain Poisson's ratio is usually fixed and strain compatible compression-wave velocities calculated using the strain compatible shear moduli from the horizontal component analyses combined with the low-strain Poisson's ratios. In a similar manner, strain compatible compression-wave damping values are estimated by combining the strain compatible shear-wave damping values with the low-strain damping in bulk or pure volume change. This process assumes the loss in bulk (volume change) is constant or strain independent. Alternatively, zero loss in bulk is assumed and the equation relating shear- and compression-wave damping (η_S and η_P) and velocities (V_S and V_P)

$$\eta_P \approx \frac{4}{3} \frac{V_S}{V_P} \eta_S , \tag{B-1}$$

is used.

RVT Based Computational Scheme

The computational scheme employed to compute the site response for this project uses an alternative approach employing random vibration theory (RVT). In this approach the control motion power spectrum is propagated through the one-dimensional soil profile using the plane-wave propagators of Silva (1976). In this formulation only SH waves are considered. Arbitrary angles of incidence may be specified but normal incidence is used throughout the present analyses.

In order to treat possible material nonlinearities, an RVT based equivalent-linear formulation is employed. Random process theory is used to predict peak time domain values of shear-strain based upon the shear-strain power spectrum. In this sense the procedure is analogous to the program SHAKE except that peak shear-strains in SHAKE are measured in the time domain. The purely frequency domain approach obviates a time domain control motion and, perhaps just as significant, eliminates the need for a suite of analyses based on different input motions. This arises because each time domain analysis may be viewed as one realization of a random process. Different control motion time histories reflecting different time domain characteristics but with nearly identical response spectra can result in different nonlinear and equivalent-linear response.

In this case, several realizations of the random process must be sampled to have a statistically stable estimate of site response. The realizations are usually performed by employing different control motions with approximately the same level of peak accelerations and response spectra.

In the case of the frequency domain approach, the estimates of peak shear-strain as well as oscillator response are, as a result of the random process theory, fundamentally probabilistic in nature. For fixed material properties, stable estimates of site response can then be obtained with a single run.

In the context of the RVT equivalent-linear approach, a more robust method of incorporating uncertainty and randomness of dynamic material properties into the computed response has been developed. Because analyses with multiple time histories are not required, parametric variability can be accurately assessed through a Monte Carlo approach by randomly varying dynamic material properties. This results in median as well as other fractile levels (e.g. 16^{th}, mean, 84^{th}) of smooth response spectra at the surface of the site. The availability of fractile levels reflecting randomness and uncertainty in dynamic material properties then permits a more rational basis for selecting levels of risk.

In order to randomly vary the shear-wave velocity profile, a profile randomization scheme has been developed which varies both layer velocity and thickness. The randomization is based on a correlation model developed from an analysis of variance on about 500 measured shear-wave velocity profiles (EPRI, 1993; Silva et al., 1997). Profile depth (depth to competent material) is also varied on a site specific basis using a uniform distribution. The depth range is generally selected to reflect expected variability over the structural foundation as well as uncertainty in the estimation of depth to competent material.

To model parametric variability for compression-waves, the base-case Poisson's ratio is generally fixed. Suites of compatible random compression- and shear-wave velocities are then generated based on the random shear-wave velocities profiles.

APPENDIX B

To accommodate variability in modulus reduction and hysteretic damping curves on a generic basis, the curves are independently randomized about the base case values. A log normal distribution is assumed with a σ_{ln} of 0.35 at a cyclic shear strain of 3 x 10^{-2}%. These values are based on an analysis of variance on a suite of laboratory test results. An upper and lower bound truncation of 2σ is used to prevent modulus reduction or damping models that are not physically possible. The random curves are generated by sampling the transformed normal distribution with a σ_{ln} of 0.35, computing the change in normalized modulus reduction or percent damping at 3 x 10^{-2}% shear strain, and applying this factor at all strains. The random perturbation factor is reduced or tapered near the ends of the strain range to preserve the general shape of the median curves (Silva, 1992).

To model vertical motions, incident inclined compression- and shear (SV)-waves are assumed. Raytracing is done from the source location to the site to obtain appropriate angles of incidence. In the P-SV site response analyses, linear response is assumed in both compression and shear with the low-strain shear-wave damping used for the compression-wave damping (Johnson and Silva, 1981). The vertical and horizontal motions are treated independently in separate analyses. Validation exercises with a fully 3-D soil model using recorded motions up to 0.50%g showed these approximations to be validate (EPRI, 1993).

In addition, the site response model for the vertical motions has been validated at over 100 rock and soil sites for three large earthquakes: 1989 **M** 6.9 Loma Prieta, 1992 **M** 7.2 Landers, and the 1994 Northridge earthquakes. In general, the model performs well and captures the site and distance dependency of vertical motions over the frequency range of about 0.3 to 50.0 Hz and the fault distance range of about 1 to 100 km.

APPENDIX B

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