1 of 5

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EARLY SITE PERMIT DEMONSTRATION PROGRAM

Guidelines for Determining Design Basis Ground Motions

1693

Volume I

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GUIDELINES FOR DETERMINING DESIGN BASIS GROUND MOTIONS

Early Site Permit Demonstration Program

**Prepared by
Electric Power Research Institute
Palo Alto, California**

Project RP3302

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Volume I

GUIDELINES FOR DETERMINING DESIGN BASIS GROUND MOTIONS

PROJECT OVERVIEW

Introduction

This report develops and applies a methodology for estimating strong earthquake ground motion. The motivation was to develop a much needed tool for use in developing the seismic requirements for structural designs. An earthquake's ground motion is a function of the earthquake's magnitude, and the physical properties of the earth through which the seismic waves travel from the earthquake fault to the site of interest. The emphasis of this study is on ground motion estimation in Eastern North America (east of the Rocky Mountains), with particular emphasis on the Eastern United States and southeastern Canada. Eastern North America is a stable continental region, having sparse earthquake activity with rare occurrences of large earthquakes. While large earthquakes are of interest for assessing seismic hazard, little data exists from the region to empirically quantify their effects. Therefore, empirically based approaches that are used for other regions, such as Western North America, are not appropriate for Eastern North America. Moreover, recent advances in science and technology have now made it possible to combine theoretical and empirical methods to develop new procedures and models for estimating ground motion.

The focus of the report is on the attributes of ground motion in Eastern North America that are of interest for the design of facilities such as nuclear power plants. Specifically considered are magnitudes M from 5 to 8, distances from 0 to 500 km, and frequencies from 1 to 35 Hz. The information contained in this report may be used for site screening as well as for detailed site characterization that is needed to assess the seismic safety of a structural design. The results of this report can be used to determine seismic hazards, provided the magnitudes and distances of potential earthquakes are predetermined.

This study was conducted by a team of experts in seismology, geotechnical engineering and seismic engineering. The investigations were carried out over a period of approximately 18 months from September 1991 to March 1993. A series of focused workshops was held with project participants to help achieve consensus recommendations. The project was sponsored by the U. S. Department of Energy, Sandia National Laboratories, Southern Electric International, Commonwealth Research Corporation, Public Service Company of New Jersey, and the Electric Power Research Institute as part of the Early Site Permit Demonstration Program. The project was managed by the Electric Power Research Institute.

Approach

The methodology for determining ground motion that was developed in this effort is markedly different than has been used for previous studies of Eastern North American ground motion. In this study, theoretical computational models are used to characterize earthquake ground motion that have been calibrated in various ways against data from earthquakes from throughout North America and the world. In these models, the contributions to ground motion, including its variability, are evaluated using physical representations of earthquake processes. These processes involve the initial generation of seismic energy or waves at the earthquake fault ("source effects"), followed by the propagation of seismic waves through the earth's crust ("path effects"), and finally the modification of seismic waves as they travel through soils near the earth's surface ("site effects"). Thus, for a given earthquake magnitude and distance, each of these physical processes contributes to the characteristics of the ground motion at the earth's surface:

- The source contributes character by virtue of the wide range of dynamic interactions that occur when seismic energy is generated by the rupture of an earthquake fault.
- The seismic path contributes to the ground motion's character and its variations because of effects such as reflection, refraction and damping of seismic waves within the earth's crust in response to the various physical properties along the wave path.

- The site contributes to the evolution of seismic waves in much the same way as the path, though on a smaller scale. The principal element is the variation in soil (e.g., clay, gravel, sand) properties .

It is the characteristics of the seismic source, path and site effects that form the basis for the parameters in the theoretical models.

An objective was to develop a set of relations that could be used to predict the expected ground motion on rock or on soil for a future earthquake. Using the above physically based and empirically calibrated ground motion model, a wide range of values was assigned to each of the parameters. The combination of all model parameters and their ranges of values were used in computer simulations to produce hundreds of records of earthquake ground motion for each magnitude and distance considered. While each earthquake simulation represents a possible future earthquake, each earthquake is not equally likely to occur. Therefore, based upon extensive analyses of past earthquakes and comparisons to model predictions, distributions were assigned to the values for each model parameter. The parameter value distributions were based on partitioning their variability into two types: uncertainty, which is due to our lack of knowledge of earthquake characteristics; and randomness, which is due to the inherent variability of those characteristics. Finally, the individual parameter weights were combined for each earthquake simulation to produce the appropriate "distribution" of earthquake ground motion for each magnitude, distance and frequency considered. Together these distributions constitute a family of functional relations that define the final engineering ground motion model. The engineering model defines ground motion for median levels and associated variability.

To accommodate site effects, site amplification factors were developed for a range of soil types and depths that are representative of soil conditions in Eastern North America. These factors were derived by first accumulating data that describes the seismic characteristics of various soils. These data were used to assess the variability of their seismic properties, especially the variation in wave velocity with depth. In addition, seismic velocity and damping data were gathered from three reference sites using a variety of field and laboratory techniques. These data were used to improve the physical understanding of the dynamic processes of soil response and to assess our ability to directly measure the physical properties needed to estimate

site effects. The estimation problem is particularly difficult because the seismic properties of soils change depending upon the level of shaking. The resulting "nonlinear" effects generally cause the ratio of soil to rock motions (i.e., soil amplification) to decrease as the corresponding rock motion increases. The quantification of these effects through theoretical modeling and comparisons to empirical data resulted in factors that describe the amplification of soils relative to rock for several soil categories. Amplification factors (for soil relative to rock) were developed for a wide range of rock motions and are given as median values with variability.

Finally, based upon extensive geotechnical data that were collected at the three reference sites and analyzed as part of this program, a guideline was developed for making site-specific assessments. This site-specific guideline is for use in planning and conducting a systematic and thorough geotechnical investigation of soil properties at a potential site. Guidance is also provided for the conduct of dynamic analyses required to determine the response of the soil column to earthquake shaking at (and beyond) the levels of motion of interest to the seismic design.

Conclusions

The attenuation and site amplification factors that were developed in this study can be used for screening potential sites in Eastern North America. This can be done prior to conducting extensive site investigations. The application of these procedures to site screening requires that information be available regarding earthquake magnitudes, distances and certain site properties (e.g., soil depth and site geology). The magnitudes and distances of potential earthquakes may be derived either probabilistically or deterministically. The site-specific guideline together with the results of investigations of the three reference sites provides the needed background information to conduct an appropriate geotechnical and seismic engineering investigation of a site. This guideline is appropriate to investigations of a wide range of site conditions and soil depths. While there are certain soil types (e.g., those with liquefaction potential) for which this guideline is not appropriate, it may be used generally both within and outside Eastern North America..

The information compiled in this report represents a comprehensive assessment of the nature of earthquake ground motion in Eastern North America. The results of this study will be useful in seismic hazard evaluations and in establishing seismic design standards for many years. The results incorporate best estimates of ground motion and the randomness and uncertainty associated with those estimates for a wide range of earthquake magnitudes, distances and frequencies.

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LIST OF ACRONYMS

ASTM	American Society of Testing Materials
CDMG	California Division of Mines and Geology
CUREE	California Universities for Research in Earthquake Engineering
ECTN	Eastern Canadian Telemetered Network
ENA	Eastern North America
EUS	Eastern United States
FSAR	Final Safety Analysis Report
LSTC	Large-Scale Triaxial Chamber
NCEER	National Center for Earthquake Engineering Research
NEIC	National Earthquake Information Center
OCR	Over Consolidation Ratios
ORI	Ohsaki Research Institute
PSAR	Preliminary Safety Analysis Report
RCTS	Resonant Column and Torsional Shear
SPT	Standard Penetration Test
UNH	University of New Hampshire
WNA	Western North America
WUS	Western United States

SECTION 1
INTRODUCTION

INTRODUCTION

1.1 Background¹

This report summarizes work to investigate earthquake ground motions in eastern North America (ENA), east of longitude 105°W. The general purpose is to develop engineering estimates of earthquake ground motions for application in the eastern United States (EUS) based on a physical understanding of how the earth behaves. explicitly expressing uncertainties in estimates caused by lack of knowledge both about properties of the earth and about physical processes that govern the generation and transmission of seismic energy.

In the assessment of suitability for future nuclear power plant siting evaluations there is a need for both a generic ground motion model for site screening and a set of procedures for site characterization for development of a site-specific design basis. The generic model yields the first-order estimate to the suitability of a site and is manifested by a set of equations with a limited number of parameters appropriate to the source, path and site combination. The equations are conditional on a measure of earthquake size (generally a magnitude measure) and distance from the energy release to the site. The earthquake size measure (typically body-wave magnitude determined from L_g waves, m_{L_g} in the EUS) must be an appropriate measure of energy released in the engineering band of interest. The function of distance must account for the wave types that transmit seismic energy through the earth (both body and surface waves) and must account for intrinsic attenuation of energy in the earth, both along the entire travel path and in the near surface crustal layers where attenuation may be especially prominent.

Next, for development of a site-specific design basis, the site applicant is generally

¹ Refer to Appendix 1.A for the Glossary of Seismological and Geotechnical Terms.

required to obtain site-specific data. In particular, local geological and geotechnical data need to be obtained and analyzed to determine the appropriate site response. The site response is generally considered to be the evolution of the seismic wave field from bedrock below the site to the top of the soil column, or free surface of the earth. In the past 10 years there has been a significant evolution in methods to acquire site-specific data--particularly the definition of soil properties -- and methods to calculate site response, especially for large seismic inputs.

It is anticipated that the current recommendations for earthquake ground motions will be robust with respect to changes in our understanding of ground motion in the EUS. The field of strong-motion seismology changes slowly, most often as the result of new information gained from earthquake observations; and given that earthquakes are sufficiently rare events in the EUS, significant advances in our understanding of the earthquake ground motion process are unlikely in a shorter time frame. This is not to say that the field is stagnant, or that we know all we need to know to estimate the shaking during future earthquakes with precision. On the contrary, there is much to be learned, both about the transmission of seismic waves in the earth and about properties of the earth in many locations that affect that wave transmission. Rather our knowledge about processes and properties of the earth is unlikely to change greatly in the next few years, over most of the EUS. In addition, the specific focus of this project is on quantifying the variability in earthquake motions that may be anticipated given today's information base and physical understanding of earthquake motion generation and transmission. We therefore expect that additional information will allow us to reduce the estimation uncertainty and provide more precise ground-motion estimates.

Thus there is great value in the collection of data in a region or at a site, to reduce uncertainties on the values of parameters used in ground motion estimation. The procedure developed in this project allows for the reduction of uncertainties if data of certain types are collected. The power of the recommended method is that it allows

ground motion estimation with different amounts of available data, so the method can be used for screening sites (when little or no site- or vicinity-specific data are available), for ranking candidate sites (when some data may be available), and for final analysis and design at a specific site (when very detailed, site- and regional-specific data have been collected). This flexibility means that designers and regulators can proceed with decisions with a knowledge of how uncertainty in ground motion estimates might change as new data are collected and incorporated into the estimation process. This will bring stability and efficiency to the design and licensing process. Thus the procedure recommended here for estimating strong shaking should be evaluated as a method applied for the next few years in the eastern US. We simply do not see any development that would change these recommendations, other than the unlikely occurrence of several moderate to large EUS earthquakes that are well recorded from near-source to teleseismic distances on a variety of soil conditions.

The procedures for estimating strong shaking have been developed using multiple interpretations and assumptions. Each of these interpretations and assumptions might be applicable over all of the EUS, or several might apply, each to a specific region. The incorporation of multiple assumptions is made explicitly to give stability to the estimation process; it is unlikely that any one of these assumptions will be proven to be the "correct" model during the next few years. This procedure is preferred over a model that adopts a single set of assumptions, where that set of assumptions must be updated following every data set collected from an EUS earthquake.

This methodology follows the philosophy that it is best to make mean-centered estimates of ground shaking, taking all uncertainties into account. There is no attempt to be "conservative" in the sense of adopting higher-than-average assumptions as best estimates. This will allow more quantitative, explicit statements of safety than if conservatism is introduced during several steps of the process and

compounded resulting in an unknown degree of seismic safety.

Engineering interest in seismic ground motions for nuclear power plants (NPPs) is in the frequency range 1 to 25 Hz, so this procedure estimates response spectra amplitudes for those frequencies, plus peak ground acceleration (PGA). The last parameter is included for historical reasons, so that this procedure can be compared to others that have been published, but is not necessary for the estimation of response spectra, as spectral amplitudes are estimated directly.

1.2 Previous Models of EUS Ground Motions

Previous procedures used to estimate earthquake shaking in the EUS span a range of methods. The earliest methods, used prior to the 1980s, relied on estimating the attenuation of Modified Mercalli intensity (MMI) from observations of historical earthquakes (e.g., Gupta and Nuttli, 1976; Bollinger, 1977) and converting MMI estimated at the site to peak motion parameters such as PGA (e.g., Trifunac and Brady, 1975; Murphy and O'Brien, 1977). Spectra were then estimated by scaling a standard spectral shape by the peak parameters (Newmark et al., 1973).

The first method of estimating ground motion in the EUS on a physical basis was developed by Nuttli (1979). He estimated the Fourier spectrum of motion as a function of earthquake magnitude and distance based on seismological theory, and used a simple theoretical scaling to estimate peak time-domain amplitudes from the Fourier spectrum. Inherent in this method were relations that scaled the corner frequency of the motion by the seismic moment of the earthquake.

Atkinson (1984) applied random vibration theory (RVT) to estimate earthquake motions in ENA in 1984. This was followed by more detailed applications (Boore and Atkinson, 1987; Toro and McGuire, 1987) that included detailed comparisons of estimates with data. RVT allows a complete estimate of ground motion characteristics, including frequency content, duration, peak time-domain amplitudes, and seismograph response (from which measures of earthquake size on different magnitude scales can be derived) (see, for example Hanks and McGuire, 1981, and

Boore, 1984). The RVT method has been verified in California over a wide range of magnitudes and distances (e.g., McGuire et al., 1985), and its adoption for predictions in the EUS was a natural extension of the success of the method to earthquakes in California.

All of the aforementioned studies presented single models for estimating EUS ground motions. Two major efforts to quantify *uncertainties* in predicted ground motion amplitudes were undertaken by EPRI/SOG (1988) and Lawrence Livermore National Laboratory (LLNL, 1987) in their respective projects to characterize earthquake hazards at NPP sites. The EPRI/SOG study quantified uncertainties by making predictions for three sets of assumptions, including two based on RVT and one based on the Nuttli approach of predicting peak motion parameters and scaling a standard spectral shape. For site amplification, the EPRI/SOG study developed soil categories representative of the range of site conditions of operating nuclear power plant sites in the eastern US, as appropriate for one-dimensional site response analyses using Random Vibration Theory. The soil categories developed were based primarily on shear-wave velocities as a function of depth appropriate for the dominantly stiff-soil sites encountered in the EUS. The uncertainties associated with site amplification factors were not well-constrained, because of poor constraints on dynamic soil properties and on analysis methods. The LLNL study polled and quantified the opinions of five ground motion consultants on the range of estimates of EUS ground motions, and quantified ground motion uncertainty using the weights ascribed to those methods by the participants. For site amplification the LLNL study calculated soil response for several categories of sites using one-dimensional shear wave models with random input motions. Figures 1-1 and 1-2 show comparisons of the mean ground motion estimates for rock outcrops from the equations developed by the EPRI/SOG study and those adopted by the LLNL study; these ground motion estimates generally show similar amplitudes and range.

As broad as the EPRI/SOG and LLNL studies were, both had deficiencies. Neither

made a detailed study of the physical processes driving ground motion characteristics but adopted methods that were readily available at the time. The EPRI/SOG study did not use a wide range of expert opinion, but relied on one group to make judgments and interpretations, and included one model that scales spectral amplitudes from peak motion parameters using California spectral shapes, which are deficient in the high-frequency content typical of EUS ground motions. This is now judged to be an obsolete method. Some of the LLNL participants used this method. Some also gave significant weight to methods that estimate ground motion by predicting a site MMI and converting that site MMI to a response spectrum using correlations from California, a procedure that is also obsolete. The obsolescence of methods that scale California spectral shapes, and of those that convert estimated site MMI, comes from two recent advances. First was the recognition in recent years (e.g., Atkinson, 1984; Boore and Atkinson, 1987; Toro and McGuire, 1987) that the band-limited, white noise model of ground motion is an accurate, quantitative method to characterize shaking in the EUS. Second was the collection of digital seismograph data over a range of earthquake magnitudes and distances, which could be used to estimate parameters in a stochastic model. These advances came subsequent to the EPRI/SOG and LLNL efforts, so neither study is satisfactory as a fundamental basis on which to license NPPs over the next five to ten years. Further, studies of ground motion in the EUS since these two studies were published (e.g., Burger et al., 1987; Barker et al., 1989; Somerville et al., 1990; Atkinson and Mereu, 1992) have focused on specific characteristics of shaking in a subregion, not on a broad methodology of ground motion estimation that encompasses all major uncertainties and that is applicable throughout the EUS.

In summary, a study of the type undertaken here has never been performed for the EUS. Previous studies either have concentrated on specific physical characteristics of ground shaking, or have quantified uncertainties but have included methods and equations that were obsolete and not based on a physical understanding of how seismic energy is transmitted through the earth.

More recent research has provided new data on many of the input assumptions to the ground motion model, including those concerning attenuation (Atkinson and Mereu, 1992), duration (Atkinson, 1993) and, most significantly, the source (Atkinson, 1993). The implications of the new data in these areas is considered in the current study.

1.3 Seismic Hazard Perspective

The EUS is an area of generally low historical seismicity, with a few moderate-to-large earthquakes that, if they recurred today in a populated area, would cause significant damage. Among these are the earthquakes in 1811-1812 near New Madrid, Missouri (magnitudes estimated around 8), the 1886 Charleston, South Carolina earthquake (magnitude estimated around 7), the 1929 Grand Banks earthquake (magnitude $m_b = 7$) and the 1925 Charlevoix earthquake ($m_b = 6.5$) (see, for example, Johnston et al., 1993). These events, and the more numerous earthquakes with magnitudes in the 5 to 6 range, are sufficient cause for taking precautions against seismic motions in the design of NPPs. There are large uncertainties in the causes and characteristics of earthquakes in the EUS, and the situation is complicated because earthquakes may have several causes in one region, and several other causes in another region. Thus what we learn in one region about earthquake mechanisms and source dynamics may or may not apply to other regions of the EUS. Much effort has therefore gone into documenting these uncertainties as we understand them today (e.g. in the EPRI/SOG and LLNL studies).

Because of these significant uncertainties on the causes and characteristics of earthquakes, the concept of a deterministic "design earthquake," in the sense of a worst-case event, is lost. The largest, closest earthquake can only be defined in the context of a tectonic hypothesis, which itself may have high, or low, or exceedingly low credibility. Thus the worst-case earthquake may only have meaning in the context of a probabilistic statement. Further, the worst-case earthquake may be different for high frequencies of the ground motion than it is for low frequencies.

Finally, even when earthquake source properties (e.g. magnitude, location, stress drop, and style of faulting) are specified, there is considerable variation of ground motions from site to site at the same distance, so the concept of a maximum earthquake only makes quantitative sense in a probabilistic context.

A probabilistic seismic hazard assessment (PSHA) is intended to integrate over all possible earthquakes in a region, accounting for their relative probabilities of occurrence, to derive probabilities of various levels of ground shaking occurring at a site in a future time interval. Modern PSHAs account explicitly for uncertainties in earthquake characteristics and in ground motion characteristics, so the deliberate evaluation of ground motion uncertainties and their effects on ground motion prediction is critical. With such information, PSHA can indicate which earthquake magnitudes and distances dominate the hazard at a site for a given frequency and probability level. This can help to focus on ground motion prediction for the right characteristics, and also to help design engineering facilities for the most likely characteristics of earthquake shaking at any probability level.

It is useful to illustrate this point for typical sites in the central and eastern United States and for various measures of ground motion. Figure 1-3 shows the location of four nuclear-plant sites selected for this analysis. Calculations were performed using the EPRI/SOG seismicity interpretations and attenuation functions (for the sake of expediency only the median EPRI/SOG attenuation equation was used). The Millstone and Limerick sites are located in areas of relatively moderate to high local seismicity, with maximum magnitudes typically lower than m_{Lg} 6.5. These sites are typical of the eastern seaboard. The Clinton and Zion sites are located in areas of relatively low seismicity and low maximum magnitudes, at distances of 300 to 600 km from the New Madrid seismic zone. These sites are typical of midwestern sites within a 600 km radius from the New Madrid zone, which has the highest rate of seismicity in the United States Stable Continental Region.

Figures 1-4 and 1-5 show the contribution to the hazard for peak ground acceleration (PGA), by magnitude and distance, for the Millstone and Clinton sites. Most of the hazard comes from magnitudes lower than 6.5 and distances smaller than 50 km.

Figures 1-6 and 1-7 show the same type of results for 1-Hz spectral velocity. This relatively low-frequency measure of ground motion differs from PGA in that it has a stronger dependence on magnitude and a somewhat weaker dependence on distance. Also, the results are very different between the two sites. For Millstone, the contributions to the hazard are shifted to somewhat higher magnitudes and longer distances. For Clinton, nearly all the hazard comes from large earthquakes in the New Madrid seismic zone.

These results are useful for identifying which parameters in ground-motion models deserve the most attention from an engineering standpoint. Because high-frequency (>5Hz) ground motions are most important for nuclear-plant structures and equipment, the most effort should go into refining ground-motion estimates for magnitudes 5.0 to 6.5 and distances less than 50 km. Issues such as stress drop variability, site response, and vertical ground motions (for certain structures), are important in this magnitude-distance range.

Though somewhat less critical than high-frequency ground motions, ground motions at frequencies of 1 to 5 Hz are still important for certain nuclear-plant structures (and for equipment mounted on those structures, see Toro et al., 1989). Significant effort should go into refining low-frequency ground-motion estimates from large earthquakes at distances up to several hundred kilometers. Issues such as source scaling, Lg wave propagation, and anelastic attenuation (for frequencies near 1 Hz only) are important in this magnitude-distance range.

Considerable emphasis is placed in this study on the development of well-quantified soil amplification factors and in the development of guidelines for geotechnical site

characterization. Because of significant advances in methodologies in this area, it is evident that large benefits in reducing conservatism in ground motion estimation are gained through investigation of site properties.

1.4 Regulatory Perspective

The Standard Review Plan of the USNRC (1990) for vibratory ground motion identifies a hierarchy of approaches considered appropriate for the estimation of seismic ground motion at a site:

1. The direct use of recorded ground motion records to develop a statistical basis for estimation of design response spectra from multiple records of similar magnitudes, distances, and site conditions;
2. The use of scaled, recorded ground motion records that have been adjusted to account for differences in magnitude, distance, and site conditions from those to be used for design.
3. Estimation of peak motion parameters (peak acceleration, velocity, and displacement) using *state-of-the-art* relationships appropriate for the region of study, and development of response spectra using standard spectral shapes scaled to those peak parameter values. Estimating motions by using correlations between ground motion parameters and MMI is discouraged because of the lack of data at the higher intensities.
4. The use of theoretical-empirical modeling of ground motion, when the input parameters and the appropriateness of the model are well-documented.

The development of this hierarchy was based on the underlying assumption that earthquake ground motions in the EUS and in California are similar, at least in the near field. Recent recordings however have shown that motions in the two regions

differ, so it is not appropriate to use empirical data from California to represent EUS ground motions. Further, the direct empirical observations in the EUS at high levels of ground motion are too sparse to justify a data-based approach in that region.

For these reasons, it is necessary to use the theoretical-empirical modeling method in the EUS. This approach uses a theoretical model to estimate ground motion amplitudes in the frequency band of interest to engineering analysis and design, a model that has been validated using data largely from California where instrumental records are available over a wide range of magnitudes and distances. The model is adjusted to reflect properties of the earth in the EUS, and is checked to verify that it is accurate at low levels of shaking in the EUS where instrumental records are abundant. Thus there is confidence that the extrapolation to higher levels of ground motion is reasonable, within the uncertainties documented by the model. In fact, there is no other defensible way to estimate ground motions for the region; the use of MMI introduces uncertainties that can be neither quantified nor resolved because of the observational, subjective, regionally-dependent nature of the scale, and the use of peak motion parameters requires adopting spectral shapes from California that may be deficient at certain frequencies. Moreover, peak motion parameters reduce information about the ground motion (such as duration and frequency content) to one or two values, thus greatly restricting the information available.

The theoretical-empirical approach adopted here is the most appropriate method to estimate earthquake shaking in the EUS. It corresponds to points 2 and 4 in the above hierarchy, where for point 2 the scaling is accomplished by theoretical methods that have been shown to be accurate by comparison to data. The theoretical-empirical method is preferred because there are no direct instrumental records of ground motion in the EUS at the damaging levels of shaking corresponding to typical NPP seismic design levels.

In addition to the hierarchy on preferred approaches, the Standard Review Plan

(1990) identifies attributes that are judged to be important to any ground motion model, with the highest acceptability given to the model with the most attributes. These are:

1. ability to describe the full response spectrum,
2. ability to provide a full description of the uncertainty (median, mean, and fractiles),
3. ability to account for site effects,
4. ability to estimate time histories of motion,
5. ability to estimate duration of shaking and number of strong motion cycles,
6. ability to define ground motion at different elevations, and
7. ability to incorporate effects of material property variations.

The model developed here has all of the above attributes, and in fact goes further than any previous model in its ability to provide a complete description of seismic ground motion and its uncertainty for analysis and design.

Currently, the only regulatory guidance for geotechnical site characterization is given in R.G. 1.132 (USNRC, 1977), "Site Investigations for Foundations of Nuclear Power Plants". This guidance is limited to a general description of the types of geotechnical investigations that are recommended for assessment of foundation stability. First, considerable advances have been made in these methods in the past 15 years that warrant including in new guidance criteria. Second, R.G. 1.132 does not bridge the gap from site geotechnical issues in foundation engineering to seismological issues in determining the design basis ground motion. A coordinated effort is needed to generate optimally a set of geotechnical data (a) for input to a dynamic analysis of site response, and (b) for estimation of the design basis ground motion. Therefore, Chapter 7 of the Site Effects Report develops a complete guideline for consideration as a new regulatory guide on site response analysis and geotechnical site characterization. The guideline draws extensively on lessons learned from the

development of test cases as well as on extensive review of recent literature.

1.5 Significant Ground Motion Issues

Several issues are perceived as significant to the estimation of seismic shaking in the EUS. They may or may not be significant in the estimation of seismic hazard at a particular site, depending on the areal sources, magnitudes, and distances affecting that site, and depending on the frequency range of engineering interest.

1.5.1 Source Issues

Information about seismic source characteristics of eastern North American earthquakes is available from teleseismic and regional seismograms of all of the larger events that have occurred since the 1920s. The information available at longer periods (1 second and longer) is sufficient to make comparisons of this set of earthquakes with those of other stable continental interiors and with those of tectonically active regions. Recent analyses indicate that the Brune stress drop (Brune, 1970, 1971) of these events, which reflects the overall duration of the source rupture and the average rate of release of seismic moment, is about 100 bars on average (Somerville et al., 1987; CERI, 1992; Boatwright and Choy, 1992). However, when we wish to prescribe source characteristics at frequencies above 1 Hz, which are relevant to the seismic design of nuclear power plants, our data base is much more sparse because we have few on-scale high-frequency seismograms of these larger earthquakes. To extrapolate our teleseismic Brune stress drop estimates from these larger earthquakes to higher frequencies, we need a model of the spectral shape. There is evidence that a spectral model having two corners (Atkinson, 1993) may be more appropriate than a Brune model having a single corner. We would like to resolve which model is more appropriate, and if this cannot be clearly resolved then we need to know whether these different spectral models could produce significant differences in predicted ground motion amplitudes. Some further constraint on spectral amplitudes at high frequencies comes from data from smaller magnitude earthquakes, but to extrapolate to the larger earthquakes of interest

requires knowledge of spectral scaling with magnitude.

Strong ground motions are influenced by a large number of aspects of source finiteness in addition to the Brune stress drop. As the size of the fault rupture increases, we expect that the rate of attenuation of strong ground motion as a function of closest distance to the rupture will tend to decrease at close distances, producing a magnitude-dependent shape. Also, the finiteness of the fault will produce azimuthal variation of ground motion amplitudes at a given closest distance due to the orientation of the fault (e.g. foot wall/hanging wall differences), and the geometry of rupture initiation and evolution on the fault plane (rupture directivity). An additional source of variation is due to the radiation pattern of the earthquake. In order to fully incorporate the uncertainty in predicted ground motions, variations in these source parameters need to be considered.

The size of the earthquake is characterized by seismic moment in source models used to predict ground motion attenuation. However, in the earthquake catalog for the eastern United States, seismic moments are generally not known for the older and smaller events. In this situation, we need either to convert L_g magnitude m_{L_g} estimates to seismic moment, or to develop a procedure for characterizing source size in terms of m_{L_g} in the ground motion model. Empirical relationships between seismic moment and m_{L_g} have been developed, but are subject to some uncertainty due to the small numbers of events that constrain the relationship at large magnitudes, and large uncertainties in some seismic moment estimates, especially those derived from spectral analysis. Relationships have been developed between seismic moment and m_{L_g} based on ground motion models, but it is necessary to demonstrate that relationships derived solely from the model are compatible with empirical data.

1.5.2 Path Issues

Traditionally, ground motion attenuation relations have been developed using regression of recorded ground motion data. In this approach, the only parameters

relating to the path between the seismic source and the recording site have been the length of the path and in some instances the focal depth, and it has not been necessary to inquire into the physics of wave propagation in the earth's crust. Recorded ground motions in the EUS show that there are clear differences in path effects in this region from path effects in California. This is illustrated in Figure 1-8, which compares recorded ground motions from the Whittier, California earthquake and the Saguenay, Ontario earthquake, both about magnitude 5.8. While there are no close data for Saguenay, recordings in the distance range 50 to 100 km clearly indicate differences in amplitudes for the high frequencies. Unfortunately the EUS ground motion data set is not large enough for the empirical approach to be effective in resolving regional variations, effects of focal depth, near-surface effects, and other factors, so it is necessary to use physical models of seismic wave propagation to guide the development of attenuation relations.

In the simplest wave propagation models, in which crustal structure is modeled by a plane layered medium, the gross crustal structure controls the geometrical spreading of waves from the source and hence the shape of the attenuation curve. At close distances, ground motions are dominated by the direct shear wave, whose geometrical spreading is controlled by the velocity gradient above the source. At larger distances, ground motions are dominated by diving rays, and the depths and velocity gradients in the lower crust control the geometrical spreading of these diving rays that are critically reflected back to the surface. We therefore expect the attenuation of ground motion in the distance range of several tens of kilometers to 200 km from the source to be primarily controlled by the earthquake source depth and by the seismic velocity and attenuation structure of the crust. Ground motion attenuation in this distance range is expected to vary with regional variations in plane layered crustal structure models. However, the lateral variations in crustal structure implied by these regional variations may cause departures from the predictions of plane layered models. Also, the scattering that is evident in recorded data is not incorporated in simple wave propagation models, other than by the use of models

with quality factor Q , which accounts for both intrinsic absorptions and scattering. The presence of scattering in recorded data will also cause departures from the calculations of simple wave propagation models.

Before we use seismic wave propagation models to develop attenuation relations, we need to establish whether there is evidence that wave propagation models provide an adequate explanation of observed ground motion attenuation. This issue can be addressed by the modeling of recorded data and the use of quantitative comparisons between recorded and model results. Since model calculations of ground motion attenuation will result in parametric dependencies on crustal structure (including Q) and focal depth, and perhaps other source parameters, further confirmation of the model can be obtained by testing whether these dependencies are observed in recorded data.

Since the effects of geometrical spreading, anelastic absorption, and scattering all contribute to the shape of the attenuation curve, there are tradeoffs involved in the estimation of each from a set of empirical data. Since the effect of anelastic attenuation is relatively small inside 100 km, this tradeoff should not result in a great deal of uncertainty in the major effects of geometrical spreading, such as the effect of critical reflections from the lower crust. However, at distances beyond 100 km, the effects of geometrical spreading and anelastic absorption become more comparable and more difficult to separate. In this situation, it is usually necessary to make certain assumptions about the form of the anelastic absorption function or the rate of geometrical spreading at large distances or both in order to estimate the parameters describing these two effects (Atkinson and Mereu, 1992).

A variety of computational procedures are available for representing the effects of wave propagation in a layered crust on ground motion attenuation, most of which require some simplifying assumptions and approximations. These include the frequency-wave number integration method, the generalized ray theory method, and

the geometrical ray theory method. We would like to select a method that is relatively simple to use but that avoids approximations and simplifying assumptions that could cause significant bias and uncertainty in predicted ground motion attenuation.

For practical application, we need to identify those regional variations in crustal structure and focal depth that can have a significant effect on ground motion attenuation, especially those for which additional modeling or data can reduce the uncertainty, or change the mean estimate, from those of previous models.

1.5.3 Site Issues

In the development of an integrated site-response methodology, the task is to account for the most significant dynamic properties and geometrical wave propagation effects in a generic procedure. This procedure needs to be complex enough to provide realistic mean estimates and uncertainties of site response, yet simple enough to be used efficiently and effectively in a wide variety of applications. In this context, there are a number of important site effects issues that need to be addressed. These are: 1) the adequacy of a vertically propagating wave model; 2) the applicability of equivalent-linear and nonlinear dynamic soil computational models; 3) the comparison of in situ versus laboratory-derived dynamic properties; 4) measurements of damping and the effects of high confining pressures (at deep soil sites); and 5) the extension of site response models from horizontally polarized S waves to vertical motions.

The difference in spectral content resulting from soil-site effects is dramatic, as illustrated in Figure 1-9. This compares response spectra recorded at two Gilroy stations, rock and soil, during the 1979 Loma Prieta earthquake, and illustrates the degree to which 1-D site conditions (characteristics of dynamic material properties) control strong ground motions. Superimposed upon these effects, for linear systems, are the effects of lateral heterogeneities upon strong ground motion. Such laterally

varying structures as surface topography, dipping interfaces, and changes in material properties contribute to two- and three-dimensional (2-D and 3-D) aspects of ground motion estimation. These non-homogeneous effects, resulting from scattering, focusing, and mode conversions are present to a some extent at all sites. In some cases, such effects can significantly alter the spectral content of ground motions as well as increase the duration of strong shaking. Results of a previous EPRI study have shown that in the bandwidth from several seconds to about 15 Hz approximately 75% of the power that is observed at the surface of sites can be modeled as vertically propagating shear waves (Silva et al., 1988). The remainder must be essentially accounted for in the effects mentioned above. The ground motion model presented in this report is based on a 1-D model of wave propagation. This means that the remainder of the ground motion effects must be treated as random, that is, they contribute to variability but are not explicitly modeled.

As levels of strain increase in soils, the constant relationship between applied stress and resulting strain is no longer valid, resulting in nonlinear soil response. While the effects of nonlinear soil response are reasonably well understood, and different modeling approaches are well calibrated in the range of strong motion observations, models can differ substantially in predicted behavior where well-calibrated data are sparse. A simplified approach to treating this condition is to approximate the response in an equivalent-linear fashion, whereby the equivalent shear modulus and damping are approximated at each strain level and the final response is determined in an iterative fashion. Nonlinear and equivalent-linear codes need to be tested to determine their ranges of applicability and sensitivities to various assumptions of material properties.

In the absence of permanent deformations, the estimation of site response is controlled primarily by the shear modulus and damping of the soil column. These parameters may be constrained by a combination of direct measurements of earthquake motions together with in situ and laboratory testing. All have limitations

which must be considered in the analysis of data for site response modeling. In particular, dynamic properties needed to predict nonlinear effects are extremely difficult to obtain in situ; therefore laboratory procedures have been developed to measure properties from intact borehole samples. Due to unquantified effects of sample disturbance and the physical limitations of extrapolating from small samples to an entire site, the results of these measurements must be treated cautiously. Where possible it is essential to make comparisons between more direct in situ measurements and analyses of strong motion records. To this end, this report considers several test cases where strong motion data and in situ and laboratory measurements are compared through a variety of analyses.

A significant source of uncertainty in dynamic analyses is the effect of confining pressure on damping; this is extremely important in dynamic analyses of deep soil sites (depths greater than a few hundred ft), where the effects of damping dominate the estimation of amplification (or deamplification). An important contribution of this report is the estimation of damping at high confining pressure. This is accomplished by measuring the effects of confining pressure directly in the laboratory and comparing the low-strain results to damping analyses of bore hole seismic data. Corroboration of these data provides the needed constraints for extending the results to high strain using strong motion data and nonlinear or equivalent-linear models.

The discussion above has focused entirely on horizontal motions generated primarily by S waves. Extension of the methodology to vertical motions is needed, especially at near-source distances. It is commonly assumed that vertical motions are approximately 2/3 horizontal at all source-receiver distances. Growing evidence suggests, however, that vertical motions often exceed horizontal motions at near-source distances (e.g., Niazi and Bozorgnia, 1992). This phenomenon is probably due to non-vertically incident S-wave energy being converted to P waves, which then propagate through the soil column with much lower damping than S waves, particularly at large strains. Under these circumstances, the extent to which vertical

motions depart from the 2/3 vertical/horizontal ratio needs to be considered in the design basis. Therefore, in the site response methodology are developed site amplification factors appropriate for vertical motions.

1.6 Organization of this Report

This report summarizes work undertaken to understand and estimate strong earthquake shaking and its uncertainty in the EUS. Section 2 describes data available from the region with which to test and calibrate models of shaking. This is a very general model that has been shown to be applicable to a wide range of earthquake ground-motion estimation problems. Most of these data are for small magnitudes or large distances and do not fall under the category of "strong shaking", but they provide a basis to assess the credibility of the models being proposed, for the ranges of magnitudes and distances where data are available. Section 3 describes the theoretical ground motion model including wave propagation, which is based on RVT, and the model used to predict the effects of surficial soils, which is based on a vertically-propagating shear wave representation of the shaking. Section 4 describes how the seismic source is quantified for the purposes of this methodology. The representation of the energy content of the source, and how the source spectrum (stress drop and corner frequencies) changes with magnitude, are addressed here. Section 5 describes how crustal path effects are incorporated into the ground motion model, including possible reflections of seismic energy from deep layers and contributions from surface waves. Both Sections 4 and 5 describe what uncertainties are important in the estimation process and how they are handled by the recommended procedure. Sections 6, 7, and 8 represent the site effects portion of the report. Section 6 quantifies the effects of local soils in modifying the characteristics of ground shaking, again describing the critical uncertainties and how they are treated. Section 7 presents a geotechnical guideline for acquiring site-specific data and for conducting a site response analysis. Section 8 describes the development of in situ and laboratory data and analysis methods from specific test sites examined in detail in the course of this project. The final recommended procedures, described primarily

in Sections 4, 5, and 6, are pulled together in Section 9, which summarizes the recommended procedure as the "Engineering Model of Strong Ground Motion," including treatment of uncertainty. This section also includes comparisons of predictions to data and to other estimates of ground motion in the EUS. Finally, Section 10 draws conclusions from the study on the physical processes that are active, how these are treated by the recommended model, and how uncertainties are handled by the procedure.

1.7 References

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Figure 1-1: Comparison of predicted peak acceleration vs. distance for the EPRI/SOG study (the thick and medium lines) and the LLNL study (the medium and thin lines). Predictions are shown for m_{Lg} 5 and 7.

Figure 1-2: Comparison of response spectra predicted by the EPRI/SOG study (thick and medium lines) and the LLNL study (medium and thin lines). Predictions are shown for m_{Lg} 5 and 7 and a distance of 25 km.

Figure 1-3: Map showing four nuclear-plant sites used to investigate the contribution of various magnitudes and distances to seismic hazard. Also shown is the seismicity in the EPRI catalog.

MILLSTONE - PGA
1.0E-04 149.90 cm/sec**2

Magnitude	Distance					All Dists.
	0-20	20-50	50-100	100-150	>150	
5.0-5.5	30.3	25.3	0.7	0.0	0.0	56.3
5.5-6.0	10.2	23.7	2.4	0.0	0.0	36.3
6.0-6.5	0.0	5.3	2.0	0.1	0.0	7.3
<6.5	0.0	0.0	0.1	0.1	0.0	0.2
All Mags.	40.5	54.2	5.1	0.1	0.0	

Figure 1-4: Contribution to hazard at 10^{-4} annual exceedance probability, by magnitude and distance: PGA at the Millstone site.

CLINTON - PGA
1.0E-04 90.02 cm/sec**2

Figure 1-5: Contribution on hazard at 10^{-4} annual exceedance probability, by magnitude and distance: PGA at the Clinton site.

MILLSTONE - 1-Hz PSV
1.0E-04 3.86 cm/sec

Magnitude	Distance					All Dists.
	0-20	20-50	50-100	100-150	>150	
5.0-5.5	7.4	2.4	0.0	0.0	0.0	9.9
5.5-6.0	15.4	26.7	3.6	0.2	0.0	45.9
6.0-6.5	0.0	15.0	12.5	4.1	1.0	32.6
<6.5	0.0	0.0	1.0	4.7	6.0	11.7
All Mags.	22.8	44.2	17.1	8.9	7.0	

Figure 1-6: Contribution to hazard at 10⁴ annual exceedance probability, by magnitude and distance: 1-Hz spectral velocity at the Millstone site.

CLINTON - 1-Hz PSV
1.0E-04 3.00 cm/sec

Figure 1-7: Contribution to hazard at 10⁻⁴ annual exceedance probability, by magnitude and distance: 1-Hz spectral velocity at the Clinton site.

Figure 1-8: Recorded ground motions during Whittier Narrows, California, and Saguenay, Ontario, earthquakes, for peak acceleration and spectral acceleration at 10 and 1 Hz (5% damping).

Figure 1-9: Response spectra (5% damped) recorded at Gilroy 1 (rock: solid line) and Gilroy 2 (soil: dashed line) during the 1979 Loma Prieta earthquake (the log average of two horizontal components is plotted).

SECTION 2

ENA EMPIRICAL GROUND MOTION DATA

2

EMPIRICAL GROUND MOTION DATA IN EASTERN NORTH AMERICA

Analyses of empirical strong motion data have provided the bases for most of the ground motion models used in engineering evaluations of earthquake hazards throughout the world. In regions with infrequent earthquakes, such as ENA, empirical models based on recorded strong motion data must be replaced by models based on other shaking measures or by models based on calibrated theoretical/numerical techniques; the latter being the focus of the study presented in this report. What recorded ground motion data that is available for a region then provides a means of evaluating the appropriateness of the developed models. It is therefore important to collect and categorize the available earthquake ground motion data in order to make it useful for evaluating ground motion models.

This section describes the data base of ground motion recordings for ENA earthquakes compiled for this project. The compilation and categorization of the data are discussed first. The data are then examined with regression models to identify the constraints implied by the data on magnitude and distance scaling and ground motion variability. The examination of the variance in the recorded motions provides one measure of the uncertainty in estimating the ground motions from individual events in the ENA. In addition, statistical analysis of strong motion data from active technical environments (e.g., California) were provided to aid in evaluating modeling scaling results. These analyses are presented in Appendix 2C.

2.1 Data Sources and Data Characterization

The earthquake recordings compiled for this project consist of all ground motion recordings obtained on strong ground motion instruments in eastern North America and all seismographic network data from magnitude $\sim m_b$ 5.0 and greater earthquakes recorded at distances of 500 km or less. A few smaller events recorded by the Eastern Canadian Telemetered Network (ECTN) that were used to evaluate various numerical modeling studies were also included in the data base. The data bases of the National Center for Earthquake

Engineering Research (NCEER) and the National Earthquake Information Center (NEIC) were searched to identify potential ground motion recordings. All identified ground motion records were then entered into a structured data base format described in Appendix 2A. The data base contains information on the earthquake sources (origin time, location, depth, size measures, and focal mechanism), the recording stations (location, instrument housing, site geology), and the recordings (source-to-site distance, record processing parameters, peak accelerations, and 5%-damped spectral accelerations).

Figure 2-1 shows the spatial distributions of earthquakes and recording stations contained in the data base and Figure 2-2 shows the distribution of the horizontal and vertical component data in terms of earthquake magnitude, source-to-site distance, and general site classification. The compiled data base consists of 191 records recorded during 51 earthquakes at 95 sites. Approximately half of the earthquakes are aftershocks or secondary events in earthquake sequences. One hundred five of the records are multicomponent acceleration or velocity recordings and the remainder are vertical component velocity recordings that have been converted to accelerograms. Figures 2-3 through 2-10 show peak 5-percent damped spectral response for selected frequencies plotted versus hypocentral distance for four magnitude intervals.

2.2 Examination of Empirical Ground Motion Data

2.2.1 Approach

The ground motion data compiled in this study provides a means of examining magnitude and distance scaling relationships and the components of variability of peak amplitude data. The data were examined by regression analyses utilizing simple attenuation models of the form used by Joyner and Boore (1982)

$$\ln[y(f)_{ij}] = C_1 + C_2(M_i - 6) + C_3(M_i - 6)^2 + C_4 \ln(R_j) + C_5 R_j + \sum_k C_k \cdot Z^k + \eta_i + \epsilon_{ij} \quad (2-1)$$

where M_i is the magnitude of the i^{th} earthquake (either m_{Lg} or M), R_j is the hypocentral distance (or fault distance for near-source recordings of large events) to the j^{th} site, the Z^k terms are a set of dummy variables to account for various classification effects in the data, and C_1 through C_k are coefficients fit to the data. The analyses were conducted utilizing a *random effects* regression formulation (Brillinger and Preisler, 1985; Abrahamson and Youngs, 1992) which partitions the total error in Equation 2-1 into an inter-event (earthquake-to-earthquake) component, η_i and an intra-event (within earthquake) component, ϵ_{ij} . The random effects model is somewhat similar in concept to the two-stage regression technique employed by Joyner and Boore (1982) in that it separates the estimation of distance scaling of ground motion from the estimation of magnitude scaling. The method also requires that possible parametric effects will usually need to be observed in data from multiple events for their effect to be statistically significant. Brillinger and Preisler (1985) have shown that the random effects model yields essentially identical results to the two-staged regression approach in fitting the Joyner and Boore (1982) data set.

2.2.2 Examination of Scaling Relationships

Regression analyses were performed to evaluate the magnitude and distance scaling parameters of Equation 2-1. Because the focus of this study is on the development of ground motion models to evaluate the hazard from moderate-to-large earthquakes, the data used in the analysis was limited to recordings from magnitude 4 and larger earthquakes. The resulting ground motion data sets consist of 66 horizontal recordings and 132 vertical recordings. The geometric mean of the peak spectral ordinates at each frequency was used in the analysis to avoid having to account for correlation between the two components in estimating the uncertainties in the fitted parameters. The specific form of Equation 2-1 used was

$$\ln[y(f)] = C_1 + C_2(M - 6) + C_3(M - 6)^2 + C_4 \ln(R) + C_5 R + C_6 \cdot Z_{SS} + C_7 \cdot Z_{IS} + C_8 \cdot Z_{DS} \quad (2-2)$$

where the terms Z_{SS} , Z_{IS} , and Z_{DS} denote site classification codes B, C, and D described in Appendix 2A, Table 2A-2. Parameter Z_{SS} takes on the value 1 for shallow soil sites (depth

to rock < 20 m) and zero for all other site conditions; Z_{IS} takes on the value 1 for intermediate depth soil sites (depth to rock 20 to 100 m) and zero for all other site conditions; and Z_{DS} takes on the value 1 for deep soil sites and zero for all other site conditions. Table 2A-2 also classifies the rock underlying the soil deposits at the recording sites into four categories: hard crystalline rock; hard sedimentary rock; softer, weathered rock; and soft rock over hard rock. Unfortunately, the only sites in the data base underlain by other than crystalline rock are those underlain by deep soil deposits while the sites underlain by shallow and intermediate depth soils are all underlain by hard crystalline rocks. Thus the effect of deep soil could not be separated from the effect of rock type in examination of the data.

Figure 2-11 shows the fitted values of the linear and quadratic magnitude scaling terms C_2 and C_3 as a function of frequency for horizontal and vertical motions. Shown are fits using Lg magnitude, m_{Lg} , and moment magnitude, M . The linear magnitude scaling term increases with decreasing frequency, as one would expect, and exhibits values at high frequency near the theoretical value of 0.6 that one would obtain from use of the stochastic model (see Section 3). Scaling with m_{Lg} appears to be slightly more linear than with M (smaller values of C_3). The large error bars indicate that the limited empirical data does not provide a tight constraint on magnitude scaling parameters.

Figure 2-12 shows the fitted values of the geometric and absorption distance scaling parameters C_4 and C_5 as a function of frequency for horizontal and vertical motions. Again, fits using m_{Lg} and M are presented, with the two magnitude scales yielding very similar distance scaling parameters. The geometric attenuation parameter lies near the theoretical value of -1 for body waves in a whole space and shows somewhat higher values for vertical motions than for horizontal motions. The absolute value of the absorption parameter tends to increase in value with increasing frequency, consistent with most representations of the frequency dependence of Q . Again, the large error bars indicate that the limited data do not provide a tight constraint on attenuation parameters.

Figure 2-13 shows the fitted values of the site classification terms C_6 , C_7 , and C_8 . The shallow and intermediate depth terms indicate fairly uniform amplification throughout the frequency range with greater amplification for the intermediate depth deposits. It should be noted that the intermediate depth results are based on very limited data. The deep soil deposits, parameter C_8 , indicates de-amplification of high frequencies and amplification of low frequencies. As discussed above, the deep soil sites are also underlain by sedimentary rock, as opposed to crystalline rock for the other site types, and the values for scaling parameter C_8 shown on Figure 2-12 represent a combination of these two effects. The de-amplification at high frequencies may result from soil damping in deeper, presumably somewhat softer soil deposits and/or an increased high frequency absorption factor, κ , for sedimentary rocks. The fact that the low frequency amplification in deep soils is similar or slightly less than that for intermediate depth soils may reflect the effect of a lower soil/rock velocity contrast for the deep soil sites.

Figure 2-14 compares the site amplification functions computed from coefficients C_6 , C_7 , and C_8 with those computed from the analysis of strong motion data from active tectonic regions described in Appendix 2C. In general, the site amplifications computed from the two data sets are similar. The largest differences between the ENA and WNA results are for frequencies of 2 to 5 Hz for intermediate depth soils and for frequencies above 10 Hz for deep soils. The soil/rock spectral ratios shown on Figure 2-14 for the WNA data do not differentiate between intermediate depth and deep soil deposits. The majority of the WNA data are from deeper soil deposits and should conform more closely with the deep soil site amplification functions computed for the east, although analyses of soil site strong ground motion data have shown that soil depth is not a statistically significant factor in predicting ground motions for frequencies of 1 Hz and higher (Campbell, 1989). As discussed above, the de-amplification at higher frequencies for ENA deep sites may be due to differences in the rock properties between the deep soil and rock sites that are not present in the WNA data set.

The results presented in Figures 2-10 through 2-12 are generally consistent with the trends that one would expect for ground motion amplitudes based on theoretical models and examination of large sets of empirical ground motion data. Because of the generally large error of estimation of the ENA scaling parameters, the effect at any one frequency is only marginally statistically significant. But taken over a wide frequency band, the results present a consistent pattern that provide broad constraints on ground motion scaling parameters.

2.2.3 *Examination of Variance of Peak Motion Parameters*

The variance in the peak ground motion parameters is estimated by finding the corresponding variances of the error terms η_i and ϵ_{ij} , denoted by τ^2 and σ^2 , respectively. Assuming independence between the two error terms, the total variance, σ_T^2 is then obtained by the sum of the two variance components ($\sigma_T^2 = \tau^2 + \sigma^2$).

Figures 2-15 through 2-19 show the values of τ , σ , and σ_T obtained from fits of Equation 2-2 to the horizontal and vertical spectral ordinates. Smaller values of the inter-event standard error were obtained when fitting to m_{Lg} magnitude than when fitting to moment magnitude M (Figure 2-15). This is coincident with a more linear scaling with magnitude found for m_{Lg} . Similar intra-event standard errors were found for both magnitude scales (Figures 2-17 and 2-18), consistent with the similar distance scaling terms obtained. The combined total error is somewhat smaller for scaling with m_{Lg} and exhibits an decrease in variance with increasing frequency, especially for vertical motions (Figure 2-17). The total standard error is somewhat larger than that reported for WNA strong ground motion data (e.g., Joyner and Boore, 1982; Sadigh and others, 1989; Campbell, 1989, 1991). The larger values may reflect the impact of larger distances and/or smaller magnitudes in the ENA data set compared to that of the WNA, or may be inherent to ENA ground motions. These issues will be explored below.

Figures 2-18 and 2-19 show the results of two sensitivity studies. Figure 2-18 shows the effect of the number of classification parameters included in the model on the computed total

standard error. The 8-parameter model is the model defined in Equation 2-2. The 5-parameter model includes only the magnitude and distance scaling terms from Equation 2-2, thus assuming no difference among site categories in spectral acceleration. Inclusion of the site classification terms results in a reduction in the computed total standard error. The statistical significance of the reduction in error was checked by using the likelihood ratio test (eg. Seber and Wild, 1989, p. 196). The likelihood ratio test statistic, LR , is given by

$$LR = 2[LL(\theta) - LL(\theta_0)] \quad (2-3)$$

where $LL(\theta)$ is the log-likelihood computed for the fit of a specified set of parameters, θ , to the data and the test is of the hypothesis that θ equals a specified vector θ_0 . In this case, the test is that coefficients C_6 , C_7 , and C_8 are all zero. The LR statistic is approximately distributed as chi-squared when the hypothesis is true (Seber and Wild, 1989). Using the likelihood ratio test, the hypothesis that the site classification coefficients are zero cannot be rejected at any individual frequency.

The 10 parameter model included the eight parameters for Equation 2-2 plus two parameters to account for effects due to site location within the various crustal regionalization zones described in Section 5. As described in Section 5, the ENA was divided into two primary crustal regions on the basis of differences in wave propagation characteristics: a primary region consisting of the majority of ENA, and the gulf coast. A third subregion was defined as part of the primary region which may have marginally different attenuation characteristics. The empirical data analyzed come from two of these zones, the primary zone and the subregion zone. A dummy variable was added that took on a value of 0 for sites in the primary zone and 1 for sites in the subregion. A second dummy variable was added to test for effects on amplitude for cases where the earthquakes and recording sites were located in different zones. Because of the limited, scattered data, the slight reduction in total variance resulting from inclusion of these two additional parameters was found to not statistically significant.

Figure 2-19 shows the effect of fitting a bilinear form for the geometric spreading term to the data. Atkinson and Mereu (1992) have shown that the bilinear form with a slope break at 60 km provides an adequate representation of the effects of crustal reflections and changes in attenuation rates observed in the ECTN network data. As indicated by the results shown on Figure 2-19, the use of a bilinear distance attenuation function results in minimal reduction in computed standard error. This may result from the consideration of data from multiple regions with different crustal effects, or may be due to the limited data being too scattered to define a clear trend.

The empirical estimates of the variability in ENA ground motion amplitudes is derived from a data set that contains a large portion of seismographic network recordings obtained at regional distances, rather than strong motion recordings at distances usually of interest in evaluating seismic hazards from moderate size events. Appendix 2B discusses an examination of the variability in regional seismographic network peak amplitude data from ENA and WNA and a comparison with the variability of strong motion peak amplitude data. The results indicate that there does not appear to be a significant difference between the variability in network and strong motion peak amplitude data and the level of variability in peak amplitudes from network data is similar in ENA and WNA.

It has been noted that the standard error in peak ground motion data recorded in WNA decreases with increasing earthquake magnitude (e.g., Sadigh, 1983, Idriss, 1985; Abrahamson, 1988; Youngs and others, 1990). The mean magnitude for the ENA data set analyzed in this study is approximately magnitude 5, significantly lower than that of typical ground motion data sets used to develop attenuation relationships for WNA. Figure 2-20 compares the standard errors of peak horizontal spectral accelerations shown in Figure 2-17 with values computed from WNA ground motion data by Sadigh and others (in preparation) for specific magnitude intervals. The comparison indicates that the values computed for the ENA data fall within the range of values computed from smaller-magnitude WNA data.

2.2.4 Summary

The analyses of the ENA ground motion data set indicate similar trends to that observed in WUS evaluations of ground motion attenuation. The peak spectral acceleration scaling parameters estimated from the data follow expected trends and have values that one would expect from evaluations of WUS data or from the predictions of theoretical models. Because of the limited extent of the data set, it provides only very broad constraints on the form and parameters of appropriate attenuation relationships for evaluating seismic hazard in the ENA. The examinations of both strong ground motion and seismographic network amplitude data indicate comparable levels of standard error for ENA and WNA data when the effects of earthquake size on ground motion variability are taken into account.

2.3 References

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Figure 2-1 Distribution of earthquake epicenters for ground motion data in ENA data base. Earthquake size is indicated by symbol size following the indicated scale. Map scale is 1:20,000,000. Note the Nahani event is not shown as it lies well to the northwest of the map.

Figure 2-2 Magnitude-distance scatter plot of ENA ground motion data base.

Figure 2-3 Peak horizontal spectral acceleration data for 34 Hz frequency from ENA ground motion data base.

Figure 2-4 Peak vertical spectral acceleration data for 34 Hz frequency from ENA ground motion data base.

Figure 2-5 Peak horizontal spectral acceleration data for 10 Hz frequency from ENA ground motion data base.

Figure 2-6 Peak vertical spectral acceleration data for 10 Hz frequency from ENA ground motion data base.

Figure 2-7 Peak horizontal spectral acceleration data for 3.45 Hz frequency from ENA ground motion data base.

Figure 2-8 Peak vertical spectral acceleration data for 3.45 Hz frequency from ENA ground motion data base.

Figure 2-9 Peak horizontal spectral acceleration data for 1 Hz frequency from ENA ground motion data base.

Figure 2-10 Peak vertical spectral acceleration data for 1 Hz frequency from ENA ground motion data base.

Figure 2-11 Magnitude scaling terms from fit of Equation 2-2 to horizontal and vertical spectral accelerations for magnitude 4 and greater events. Error bars represent plus-or-minus one standard error confidence limits on parameter value.

Figure 2-12 Distance scaling terms from fit of Equation 2-2 to horizontal and vertical spectral accelerations for magnitude 4 and greater events. Error bars represent plus-or-minus one standard error confidence limits on parameter value.

Figure 2-13 Site classification terms from fit of Equation 2-2 to horizontal and vertical spectral accelerations for magnitude 4 and greater events. Error bars represent plus-or-minus one standard error confidence limits on parameter value.

Figure 2-14 Comparison of soil/rock spectral ratios computed from site amplification factors shown in Figure 2-13 for ENA data with spectral ratios obtained from analyses of strong motion data from WNA.

Figure 2-15 Inter-event standard error, τ , for horizontal and vertical spectral accelerations from fit of Equation 2-2 to ground motion data for magnitude 4 and greater events. Error bars represent plus-or-minus one standard error confidence limits on parameter value.

Figure 2-16 Intra-event standard error, σ , for horizontal and vertical spectral accelerations from fit of Equation 2-2 to ground motion data for magnitude 4 and greater events. Error bars represent plus-or-minus one standard error confidence limits on parameter value.

Figure 2-17 Total standard error, σ_T , for horizontal and vertical spectral accelerations from fit of Equation 2-2 to ground motion data for magnitude 4 and greater events. Error bars represent plus-or-minus one standard error confidence limits on parameter value.

Figure 2-18 Effect of inclusion of classification parameters on computed total standard error for horizontal and vertical spectral accelerations (m_{Lg} scaling). The five parameter model does not include any classification terms and the 10 parameter model includes two additional terms to account for crustal structure regionalization.

Figure 2-19 Effect of use of bilinear distance attenuation function on computed total standard error for horizontal and vertical spectral accelerations (m_{Lg} scaling). The bilinear model has a break in slope at 60 km.

f2-20

Figure 2-20 Comparison of total standard error for fit of (2-2) to horizontal spectral acceleration data with standard errors for spectral ordinates for WNA strong motion data in various magnitude bands (from Sadigh and others, in preparation).

SECTION 3
THEORETICAL GROUND MOTION MODELING

3

THEORETICAL GROUND MOTION MODELING

3.1 Introduction

This section presents the tools that are needed to assess the contributions to ground motion variability from seismic source and path effects, and that are needed for development of the ground motion attenuation relations. In this process, two approaches to modeling ground motion are used. The first, referred to as the stochastic ground motion model, is a simple and computationally efficient approach. The stochastic model was selected to make many thousands of earthquake simulations incorporating a wide range of parameters for source, site, and path effects. The results of these simulations were used to build the engineering ground motion model in Section 9. The second modeling approach, referred to as the semi-empirical ground motion model, is less computationally efficient, but more rigorous in its treatment of wave propagation effects for individual source-path combinations. The semi-empirical model was used to test the significance of crustal path effects in estimating ground motion and to assess the associated path uncertainty (see Section 5).

The two modeling methods are outlined below, including validations of the methods against strong motion recordings from several large earthquakes. These validations provide the bases for their use in modeling eastern North American ground motion.

3.2 The Stochastic Ground Motion Model

The engineering ground motion model presented in Section 9 is based on a simple physical model of the earthquake source and crustal wave propagation. The model characterizes strong ground motions as stochastic in time, with a Fourier amplitude spectrum specified by a fundamentally simple and deterministic seismological

model of the source, path, and site (Hanks and McGuire, 1981; Boore, 1983, 1986; Silva, 1991). An essential and significant aspect of the model is that, while being extremely simple, it also provides estimates of strong ground motions with reasonable accuracy. Additional, but important, side benefits arising from the model's simplicity are the natural separation of source, path, and site effects and the accompanying computational efficiency. As a result, an accurate appraisal of the effects of variability in source, path, and site parameters as well as model bias can be readily quantified.

3.2.1 Model Description

The stochastic ground motion model (sometimes referred to as the Band-Limited-White-Noise or BLWN model), in which the energy is distributed randomly over the duration of the source, has proven to be very effective in modeling a wide range of ground motion observations. Time-domain measures such as peak acceleration, peak particle velocity, Wood-Anderson magnitude, and short-period P- and S-wave amplitudes as well as frequency-domain measures such as response spectra and Fourier amplitude spectra, have been predicted with reasonable accuracy (Hanks and McGuire, 1981; Boore, 1983, 1986; Boore and Atkinson, 1987; Silva and Lee, 1987; Toro and McGuire, 1987; Silva et al., 1990; Silva and Darragh, 1993).

The ground motion model employed here uses an omega-squared (ω^2) Brune source model (Brune, 1970; 1971) with a single corner frequency and a constant stress drop (Boore, 1983; Atkinson, 1984) (Figure 3-1). The ω^2 model has become a seismological standard because of its simplicity and ability to predict spectral amplitudes and shapes over an extremely broad range of magnitudes, distances and frequencies, as well as tectonic regions worldwide. In particular, the ω^2 model has been used to satisfactorily predict spectral shapes for earthquake magnitudes from M -2 (Fehler and Phillips, 1991) to M 8-1/4 (Hartzell and Heaton, 1988), and high-frequency amplitudes and duration up to M 9.5 (Boore, 1986). It is the only source model that has demonstrably predicted ground motion amplitudes of interest to engineering applications over the entire magnitude range of the observational record.

Ground motion is assumed to emanate from the source at a point, thus ignoring effects associated with the finite spatial-temporal extent of the rupture process. In its simplest form, the model assumes waves propagate in a homogeneous, uniform

half space. The model is first described for this simple case, followed by a discussion of additional wave propagation effects that are incorporated into the complete modeling procedure. In a homogeneous, uniform half space, the acceleration Fourier amplitude spectrum, $a(f)$, where f is frequency, is given by:

$$a(f) = C \frac{f^2}{1 + (f/f_c)^2} \frac{M_0}{R} P(f) A(f) e^{-\frac{\pi f R}{\beta_0 Q(f)}} \quad (3-1)$$

where

- M_0 = Seismic moment
- R = Distance to the equivalent point source
- β_0 = Shear-wave velocity at the source
- $Q(f)$ = $Q_0(f)^\eta$, frequency-dependent quality factor model where Q_0 and η are model parameters
- $A(f)$ = Amplification factors
- $P(f)$ = High-frequency truncation filter
- f_c = Source corner frequency and
- C = $(1/\rho_0 \beta_0^3) \times (2) \times (0.55) \times (1/\sqrt{2}) \times \pi$

The effects of a velocity gradient in the crust or near-surface are accommodated through the $A(f)$ amplification factors or transfer function. C is a constant which contains source-region density ρ_0 and shear-wave velocity β_0 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 $(1/\sqrt{2})$. The product of these factors is a single constant factor 0.78.

Source scaling is provided by specifying two independent parameters, the seismic moment (M_0) and the stress drop ($\Delta\sigma$) (Figure 3-1). The seismic moment is related to magnitude through the definition of moment magnitude M by the relation

$$\log M_0 = 1.5 M + 16.1 \quad (\text{Hanks and Kanamori, 1979}) \quad (3-2).$$

The stress drop $\Delta\sigma$ relates the corner frequency f_c to M_0 through the relation

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

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 source area shear-wave velocity and with increasing stress drop, both of which may be region dependent.

In order to compute peak time-domain values, i.e. peak acceleration, peak particle velocity, and peak oscillator response, random vibration theory (RVT) is used to relate RMS calculations to peak value estimates (Boore, 1983; Boore and Joyner, 1984).

3.2.1.1 Point Source Model Parameters for Rock Sites. In a half-space model, the near-surface amplification factors, $A(f)$, account for the increase in amplitude as the seismic energy travels through lower-velocity crustal materials near the earth's surface (Figure 3-1) (Boore, 1986; Silva and Darragh, 1993). These factors depend on average crustal and near-surface shear-wave velocity and density. In applications to hard rock environments, which exist principally in eastern North America (ENA), the near surface (1-2 km) velocity gradients are small and this effect is neglected (Boore and Atkinson, 1987).

The $P(f)$ filter models the observation that acceleration spectral density appears to fall off rapidly beyond some region-dependent maximum frequency. This observed phenomenon truncates the high-frequency portion of the spectrum and is responsible for the band-limited nature of the stochastic model. This spectral fall-off 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. Hanks (1982) termed the phrase $f_{\max}$ to describe this site-dependent corner frequency. In the Anderson and Hough (1984) attenuation model, which is adopted in this study, the form of the $P(f)$ filter is taken as

$$P(f) = e^{-\pi\kappa(0)f} \quad (3-4).$$

$\kappa(0)$ is a site- and distance-dependent parameter that represents the effect of intrinsic attenuation on the seismic waves as they propagate through the crust from source to receiver (Figure 3-1). Kappa depends on epicentral distance (r) and on both the shear-wave velocity (β_r) and quality factor (Q_r) averaged over a depth of H

beneath the receiver or site. At zero epicentral distance, κ is given by

$$\kappa(0) = \frac{H}{\beta_r \overline{Q_s}} \quad (3.5).$$

The value of $\kappa(0)$ is attributed to attenuation in the very shallow crust directly beneath the site (Hough et al., 1988). Silva and Darragh (1993) suggest that the predominant kappa effects extend from the surface down to several hundred meters and possibly as deep as 1 to 2 km. The intrinsic attenuation along this part of the path is thought to be frequency-independent, but site-dependent, i.e., dependent on site geology (Hough et al., 1988). In this study, $\kappa(0)$ is assumed to be distance independent and is simply referred to as kappa. (Any distance-dependent attenuation or damping effects are absorbed in $Q(f)$ (Equation 3-1).) Kappa has been determined for several rock and soil sites representative of western North America (WNA) (Anderson and Hough, 1984; Anderson, 1986). For an average WNA rock site, a value between 0.02 and 0.06 sec is appropriate (Boore, 1986; Silva and Darragh, 1993). For eastern North America, a stable continental region with generally older and stronger crustal rocks, average kappa values range from about 0.004 sec to about 0.01 sec (Silva and Darragh, 1993). Similar models have been applied previously to the ENA (e.g., Silva and Green, 1989; Silva et al., 1989; Silva and Darragh, 1993).

The crustal anelastic attenuation from the source to just below the site is modeled with the frequency-dependent quality factors Q_f . In previous studies, the geometrical attenuation was taken as $1/R$ (or $1/\sqrt{R}$ for distances greater than about 100 km). In order to accommodate the effects of direct and supercritically-reflected waves in a crustal structure, this simple geometrical attenuation is replaced by a formulation due to Ou and Herrmann (1990) and later refined by Herrmann (Appendix 3A). The later extension represents an effort to increase the accuracy of the formulation, making it compatible with asymptotic ray theory for plane layered earth models. This method explicitly accounts for reflection, transmission, and free surface coefficients as well as some subcritical and all post-critical shear-wave arrivals. As a result, the numerical constants in the C term (Equation 3-1) are modified to accommodate the effects of a horizontally layered crust (Appendix 3A). With this technique, the geometrical attenuation and duration for direct plus post-critically reflected waves may be computed in a manner appropriate for the BLWN-RVT model.

In application, the plane-wave propagators of Silva (1976) are used to propagate SH or P-SV motion through the layered structure. To model an average horizontal component, the computed Fourier amplitude spectrum (Equation 3-1) is input as outcrop motion at the top of the source layer using the SH or P-SV propagators. Normal (SH for horizontal motion) or inclined (P-SV for vertical motion) incidence is assumed and the crust is taken as elastic with damping accommodated in the $Q(f)$ and kappa terms.

For fixed magnitude and distance, specific source, path, and site parameters are: stress drop ($\Delta\sigma$), crustal damping ($Q(f)$), crustal shear-wave velocity profile, and kappa (κ). These represent the point-source ground-motion parameters for a rock site.

3.2.1.2 Treatment of Site Effects. The parameters that influence the computed site response include the shear-wave velocity profile and strain dependent shear modulus and shear-wave damping. In order to accommodate the effects of surficial soils in eastern, central, and southeastern North America, generic site amplification factors are developed relative to hard rock conditions (Section 6) using the equivalent-linear approach described below.

To model soil response, an RVT-based equivalent-linear approach is used to propagate the point-source outcrop power spectral density through a one-dimensional soil column. RVT 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 (Schnabel et al., 1972) 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 significantly, 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. In this case, several realizations of the random process must be sampled to obtain a statistically stable estimate of site response. The realizations are usually performed by employing different control motions with approximately the same level of peak acceleration. In the frequency-domain approach, the estimates of peak shear strain as well as oscillator response are, as a result of the RVT, fundamentally probabilistic in nature. Stable estimates of site

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response can then be rapidly computed, permitting statistically significant estimates of uncertainties based on parametric variations.

3.2.1.3 Model Parameters. The source and path parameters used in the stochastic point-source model include source region shear-wave velocity and density, stress drop ($\Delta\sigma$), $Q(f)$ model, kappa, and crustal structure. Source region velocity and density are taken from the appropriate crustal model (Midcontinent or Gulf Coast, Section 5.5) for the given source depth. The median stress drop and kappa values, uncertainties, and distributions are shown in Table 3-1. The rationale for the median and uncertainty values is presented in Section 4 for stress drop and Section 5 for kappa.

Table 3-1 also shows the $Q(f)$ models for the Midcontinent and Gulf Coast crustal structures. To accommodate uncertainty in the Q_0 and η values for each regional $Q(f)$ model (mean values in Table 3-1), upper-range and lower-range $Q(f)$ models are defined (Section 5). This was done as an alternative to placing distributions on Q_0 and η and varying them independently. The mean values for the model parameters given in Table 3-1 are used to generate a set of ground motions for each crustal region as a function of distance for any specified magnitude and source depth. Dispersion about the median ground motion predictions are computed by varying the model parameters according to the distributions shown in Table 3-1. This "parametric" variability of ground motion is discussed in Section 9.

3.2.2 Model Validation

An essential aspect of any numerical modeling approach is a demonstration of the model's ability to predict recorded strong ground motions. In this section, ground motions are modeled and compared to actual recordings for the 1989 Loma Prieta, 1987 Whittier Narrows, 1985 Nahanni, and 1988 Saguenay earthquakes. (Section 3.2.4 presents a statistical analysis of these results.) These events were chosen because they are relatively well-recorded large events (M 5.8 to 6.9, Table 3-2), and either there are numerous strong motion records available at close distances (i.e., < 100 km) (Whittier Narrows and Loma Prieta) or the events are from ENA and were recorded at a few close-in sites (Nahanni) or a wide range of distances (Saguenay). It is important to note that while the model results are best constrained by the more numerous close-in stations available from the Whittier Narrows and Loma Prieta

events, data from the Nahanni and Saguenay events yield an essential calibration of the model for application to ENA.

An important initial step in the validation process is the selection of stations and associated seismic. For the stochastic model, site-specific effects for soil sites are incorporated into the computations separately (Section 3.2.1.2), with nonlinear response of soils treated explicitly in Section 6. Therefore, the emphasis in the validation is on maximizing the number of rock sites, particularly for the Loma Prieta earthquake. For rock sites, the determination of kappa values is more straightforward, and the effects of nonlinear soil response at high levels of motion are largely avoided. On the other hand, for the semi-empirical ground motion model (Section 3.3), the site-specific effects are not directly modeled so the emphasis was placed on including as many sites as is practical. The result is that for the stochastic model 61 sites were selected for the four events, while for the semi-empirical model, 104 sites were selected (Section 3.3.2). 39 of these sites are common to the two validation exercises; the comparison of validations using only the sites in common is discussed in Section 3.4.

Appropriate values for the model parameters stress drop, $Q(f)$, and kappa were estimated for each earthquake by fitting Equation 3-1 to the log Fourier amplitude spectra computed from recorded motions (Section 4.2). The use of log spectra is appropriate since short period ground motions are generally considered to be log normally distributed. Fitting on log amplitudes, however, results in model parameters that are appropriate for median Fourier amplitude predictions. Since Parseval's relation, used in the RVT process to relate power spectra to RMS time domain values, is based on a mean spectrum, the model prediction should be corrected by the mean/median ratio of the Fourier amplitude. This ratio is given by

$$mean / median = e^{\frac{\sigma^2}{2}} \quad (3.6)$$

where σ^2 is estimated from the residuals of the fit of Equation 3-1 to the Fourier amplitude spectrum.

Not all of the model parameters could be determined for each earthquake, depending upon number of stations and distance distribution of stations. In

particular, due to the limited number of stations and distances for the Nahanni and Whittier Narrows earthquakes as well as limited bandwidth for the Saguenay earthquake, magnitudes could not be resolved for these events. In order to provide more stable inversions, magnitudes were fixed to published values based upon teleseismic data. For consistency, the magnitude was held fixed for the Loma Prieta earthquake as well. In addition, a constant multiplicative term, D , was added to Equation 3-1 for each site. This term was included to reduce the coupling of unmodeled broad-band local crustal as well as site effects into the kappa values. If the assigned magnitudes are inconsistent with the data sets being analyzed, as is sometimes the case when comparing long period spectral levels from strong motion data to teleseismically determined seismic moments, the D terms provide a mechanism for adjustment. This reduces the tendency for the inversion process to place the effect of magnitude inconsistencies in the corner frequency or stress drop. The variability of the D terms is included as modeling uncertainty in the total modeling variability.

The model parameter estimation and subsequent forward modeling used both the simple $1/R$ ($1/\sqrt{R}$, $R > 100\text{km}$) and refined Herrmann (Appendix 3A) geometrical attenuation models for the Loma Prieta and Saguenay earthquakes. Since the models produced similar parameter values, the simple $1/R$ model is used for all four earthquakes to estimate the model parameter values and modeling variability.

3.2.2.1 Loma Prieta Model. The resulting parameters for the Loma Prieta earthquake are shown in Table 3-2. For crustal amplification, the velocity model given in Wald et al. (1991) was used. For the soil sites, GL2, PVA, and TRI site specific soil profiles were available and were placed on top of the Wald et al. (1991) crust. The soil sites were not used in estimating the model parameter values, as a sufficient number of rock sites were available. In the forward modeling for the variability calculation, the nonlinear response of the soils is accommodated using an RVT based equivalent-linear technique (Section 6). At these sites, the log average kappa value for the rock sites of 0.052 sec (Table 3-2) is used for the shallow crustal damping. The moment magnitude was fixed at 6.9 resulting in a stress drop of 171 bars. The $Q(f)$ model was also fixed with a Q_0 of 150 and an η of 0.60, values appropriate for western North America (Boore, 1983). Table 3-3 lists the sites used, individual kappa values, D terms, and point source distances. The distances are the closest distance to the surface projection of the top of the rupture assuming a depth

of 12 km, the location of the largest asperity (Wald et al., 1991).

A comparison between the computed 5% damped acceleration response spectra using the parameters listed in Table 3-2 and the log average of the two horizontal components at each site is shown in Figure 3-2. In general, the simple point-source provides a good match to the spectra computed from the recorded motions both close-in and at large distances. A feature of interest is the overprediction by the point source model at low frequencies, particularly at close sites. These high levels of motion impact the uncertainty as well as model bias at low frequencies.

3.2.2.2 Whittier Narrows Model. For the Whittier Narrows earthquake, the magnitude and $Q(f)$ model were constrained to M 5.9 and $150 f^{0.6}$ respectively (Table 3-2). The crustal model used is from Wald et al. (1988). For the soil sites, since site specific profiles were not available, a generic profile appropriate for sandy soils was used in place of the top layer of Wald et al. (1988a) crustal model. Recordings from both rock and soil sites are used in estimating the model parameter values with the response of soil sites treated as linear. The resulting stress drop is 165 bars (Table 3-2) with an average kappa value of 0.038 sec. Kappa values would, to first order, account for possible nonlinear effects through increased damping and higher values than would apply for linear response. Stations, distances, D terms, and site kappa values are listed in Table 3-4. Distances are closest distance to the surface projection of the rupture surface down to the depth of the largest asperity at 15.2 km (Hartzell and Iida, 1990).

Figure 3-3 shows comparisons between predicted and recorded motions in terms of response spectra for the Whittier Narrows earthquake. As with the Loma Prieta earthquake, the match is generally good at most of the sites. Again there is a tendency for the point source model to overpredict at low frequencies at rock sites (GVR and MTW).

3.2.2.3 Nahanni Earthquake. The Nahanni earthquake analyzed here is the mainshock, the only Nahanni event recorded on all three strong motion instruments. The moment magnitude was fixed at 6.9 with Q_0 and η fixed at the mean Midcontinent $Q(f)$ model (Table 3-2). A crustal structure was not used, as

little amplification is expected in propagating from the source region to the surface in this predominately cratonic crustal environment. Assuming a source region shear-wave velocity and density of 3.5 km/sec and 2.7 cgs, appropriate for the Midcontinent crustal model layer 1 (Section 5), the resulting stress drop is 86 bars. The average kappa value is 0.011 sec. Table 3-5 lists the sites, inversion parameters and distances. Approximate point-source distances are taken as closest distance to the surface projection of the top of the rupture plane to a depth of 9 km (Walt?) (Dave Boore personal communications, 1992). The site D values vary from 0.61 to 1.34, possibly reflecting a large uncertainty in the appropriate distance for the point source.

Figure 3-4 shows the comparison of recorded motions to model predictions for the three sites. Site 1 shows an underprediction for frequencies exceeding about 2 Hz with an average zero period spectral acceleration over 1g for the recorded motions. These very high levels of motion occur approximately 8 seconds into the record and may be associated with local effects. This short burst of energy does not appear at the other sites, lending support to a localized cause. It is not surprising then that this simple model predicts lower short-period motions. If the response spectra are computed using the first 7.5 seconds, the model and recorded motions show much closer agreement.

3.2.2.4 Saguenay Earthquake. For the Saguenay earthquake only the moment magnitude, 5.8, was held fixed. Although a large number of stations are available out to nearly 500 km (Table 3-6), permitting resolution of both $Q(f)$ and kappa values, the determination of magnitude (seismic moment), based upon the single-corner-frequency-omega-square model (Equation 3-1) is not reliable due to the limited bandwidth. Additionally, the mainshock exhibits strongly non- ω^2 characteristics having much more high frequency (>1 Hz) energy content relative to intermediate frequencies than is compatible with a simple ω^2 source model (Section 4), Boore and Atkinson (1992). The resulting high stress drop, 655 bars, (Table 3-2) reflects this model inconsistency. The resulting $Q(f)$ model is $361 f^{0.87}$ and represents paths in both the Grenville and Appalachian crustal structures. The average kappa value is 0.026 sec, but this value is ill-constrained because many of the ECTN data used have limited bandwidth or H/V ratio assumptions that limit the data (Table 3-6). Also, kappa determined from stations beyond 100 km tend to

have some correlation with $Q(f)$, so that the relatively higher kappa values at these distances might also reflect the fact that paths to these stations have lower $Q(f)$ values appropriate for the dominantly Appalachian structure they traverse. Table 3-6 lists the sites, kappa values, D terms, and hypocentral distances. For this earthquake, the initial inversion resulted in D terms that were all very close to 1. An additional inversion was then performed with D fixed to 1 at all sites. The nine most distant sites are vertical component ECTN stations. For these records, an H/V ratio of 1.4 was used.

In Figure 3-5 are shown the comparisons between computed and recorded motions, expressed as 5% damped response spectra. For most of the sites, the model provides a good match to the recorded motions at high frequencies due to the high stress drop. At low frequencies, the systematic departure of the recorded motions from the model predictions is evident at the sites with horizontal components. At the vertical component sites, labeled Z in Figure 3-5, the model overprediction is less pronounced. This may be a result of using a frequency independent H/V ratio. Over all of the sites, in general, the simple point-source model provides a reasonably good match to the recorded motions at high frequencies and is generally conservative at low frequencies.

3.2.3 Modeling Variability And Bias

For strong ground motions predicted from numerical modeling to be accepted for use in engineering design, it is necessary to demonstrate that the numerical modeling procedure can accurately predict recorded strong ground motions. In addition, an estimate of the variability in ground motion estimates derived from numerical modeling is needed, analogous to the variability given for empirical attenuation models. A quantitative assessment of the modeling or residual variability (Abrahamson et al., 1990) associated with the stochastic point-source model is computed here for the four events modeled in the previous section.

The total variability of the predicted strong ground motion should include all sources of variability: modeling variability, which is due to factors not accounted for in the source or wave propagation models; and parametric variability, which is due to variability in the values of source parameters for future earthquakes. Procedures are described by Abrahamson et al. (1990) for quantifying the goodness-of-fit between

simulated and recorded ground motions, and for estimating the total variability of predicted ground motions for future earthquakes by combining modeling variability and parametric variability. The partitioning of the total variability into modeling and parametric components may be different for different simulation procedures. Any model parameters that are estimated on an earthquake-specific or site-specific basis need to be considered in the parametric variations; whereas model parameters that remain fixed for all events and sites are not considered in the parametric variations. The parametric variability for the stochastic model is estimated in Section 9; the modeling variability is discussed below.

The modeling variability is derived from the comparison of observed and modeled (or predicted) response spectra. These spectra are shown in Figures 3-2 to 3-5 for the stochastic model comparisons. The modeling variability is defined as the standard error of the residuals of the log of the average horizontal 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 (61) for all four earthquakes. Dividing the resultant sum by the number of sites results in an estimate of the model variance or residual variance. Any model bias that exists may be estimated in the process (Abrahamson et al., 1990) and used to correct (lower) the residual variance.

Figure 3-6a shows the model bias versus frequency. The computed model bias is essentially zero at the 90% confidence level for frequencies above about 3 Hz. The negative model bias for lower frequencies is primarily due to the point-source tendency for overprediction at low frequencies at close sites (shown in the Loma Prieta and Whittier Narrows earthquakes response spectra) and the large overprediction at intermediate-to-low frequencies for the Saguenay earthquake response spectra.

As discussed in Section 9, the variability of ground motion can be separated into randomness and uncertainty. Randomness represents intrinsic variability and uncertainty represents variability due to our lack of knowledge. The modeling variability can also be separated into uncertainty and randomness terms. The residual standard error corrected for model uncertainty represents modeling randomness. In this case, both the model bias and the D (site) terms are uncertain

because they represent systematic misfits between modeled and recorded ground motions. Since it may be possible to reduce these systematic variations by refining the model, the effects of the bias and the variability of the D terms represent modeling uncertainty. The modeling randomness and the combined modeling randomness and uncertainty are shown in Figure 3-6b.

The overall low levels of the standard error and small bias at frequencies above about 3 Hz indicate the ability of the simple point-source stochastic model to accurately predict strong ground motions in a variety of crustal regions and site conditions. Additionally, any significant model bias exists only at low frequencies and reflects an overprediction of ground motions. These overpredictions are largely driven by the Saguenay earthquake.

3.3 Semi-Empirical Ground Motion Model

At frequencies below about 2 Hz, the principal features of strong ground motions can be modeled using deterministic models of the earthquake source. The ability of synthetic seismograms to match recorded strong ground motions at low frequencies has been clearly demonstrated during the past decade; a summary of events studied is given in Mendoza and Hartzell (1988) and Heaton (1990). However, at the higher frequencies of interest for this study, strong ground motions are highly affected by stochastic processes. The modeling procedure described below combines empirical and theoretical approaches to modeling ground motion effects in order to capture the essence of both deterministic and stochastic elements of ground motion. The combined semi-empirical model is used in this study to quantify path effects or, more precisely, contributions to ground motion variability made by wave propagation in the earth's crust (Section 5).

3.3.1 Model Description

In the semi-empirical ground motion modeling approach, near-source recordings of small earthquakes are used as empirical source functions to provide a realistic representation of effects such as source radiation that are difficult to deterministically model at high frequencies due to their stochastic behavior. Wave propagation effects are modeled using simplified transfer functions or Green's

functions that are designed to transfer empirical source functions from their recording sites to those required for use in simulations at a specific site. The details of the simulation procedure are described by Wald et al. (1988a) and Somerville et al. (1991), and are summarized in Appendix 3.B.

The procedure is illustrated schematically in Figure 3-7. The fault is divided into discrete elements, and the motions from these elements are lagged and summed across the fault to simulate the propagation of rupture over the fault surface. A stochastic component is included in the speeds of fault slip and rupture propagation to simulate heterogeneity in rupture dynamics. Large scale asperities (areas of concentrated slip and high-frequency radiation) are introduced by varying the slip distribution over the fault surface.

The Green's functions are calculated using the method of generalized rays (Helmberger and Harkrider, 1978). Rays corresponding to the direct P and S waves and to the primary reflection (and head wave beyond critical angle) from each interface below the source are included, as shown schematically in Figure 3-8. The overall seismogram is produced by summing these various rays, which generally arrive at the site at different times. The amplitude and time relationships between these arrivals change with distance, producing seismograms whose amplitudes and durations change with distance. In a homogeneous half space, the rate at which ground motion decays with distance is the inverse source distance, $1/R$. At source-site distances less than the critical distance (the distance beyond which all seismic energy is reflected back to the surface from a given boundary), the interference between the upgoing wave and waves reflected due to the velocity gradient below the source causes attenuation more rapid than $1/R$. Beyond the critical distance, however, critical reflections cause the attenuation to be less rapid than $1/R$. Thus the peak amplitudes and duration of the seismogram are a function of the crustal structure and focal depth.

The simplified Green's functions that are used, described further by Somerville et al. (1991), are the response of the crust for P, SV and SH potentials. The receiver function is represented empirically, as described below, and the radiation pattern is assigned an average value. At high frequencies, the small set of rays that used are expected to contain most of the important arrivals out to distances of 200 km. To realistically model the wave propagation at greater distances, it would be necessary

to include a much larger set of rays, or use the frequency-wavenumber integration method in place of the generalized ray method. In this application, then, ground motion modeling is not extended beyond 200 km distance.

3.3.2 Model Validation

The semi-empirical ground motion model has been validated against strong motion records of several earthquakes. These include the 1979 Imperial Valley earthquake (Wald *et al.*, 1988a); the 1987 Whittier Narrows earthquake ((Wald *et al.*, 1988b); Saikia, 1992), the 1988 Nahanni earthquake (Somerville *et al.*, 1990); the 1989 Loma Prieta earthquake (Somerville *et al.*, 1992a,b), and the 1985 Michoacan, Mexico and Valparaiso, Chile earthquakes (Somerville *et al.*, 1991).

For the purposes of this study, a validation of the method was performed using a standard set of procedures for four earthquakes: the 1989 Loma Prieta earthquake (55 stations); the 1987 Whittier Narrows earthquake (37 stations); the 1988 Nahanni earthquake (3 stations); and the 1988 Saguenay earthquake (9 stations), for a total of 104 recordings. All available digital strong motion data at source-site distances of less than about 200 km were used in the validation, regardless of site conditions. The simulation method is described in Section 3.3.1 and Appendix 3.B. The standard procedures were as follows. For each event, published slip models were used to characterize the source, and published crustal structure models were used to characterize the wave propagation path. The source and path models are described by Hartzell and Iida (1990) and Wald *et al.* (1989) respectively for the Whittier Narrows earthquake; Somerville *et al.* (1990) for the Saguenay earthquake; EPRI (1992) for the Nahanni earthquake; and Wald *et al.* (1992) for the Loma Prieta earthquake. Where these slip models included multiple time windows, a slip model was derived having a single time window. The source models of the four events are summarized in Table 3-7. The stations used in the validation of the four events are listed in Tables 3-8 through 3-11.

The empirical source functions were derived from the October 15, 1979 Imperial Valley aftershock. These empirical source functions represent a seismic moment of 0.6×10^{24} dyne-cm and a fault element size of 4km x 3km. For the Saguenay and Nahanni events, these source functions were modified for eastern North American source conditions to have a kappa value of 0.006 sec, while for the Whittier Narrows

and Loma Prieta events the source functions were unmodified and retained an average kappa value of 0.055 sec. Unlike the stochastic model, the semi-empirical model does not include the Brune stress-drop as a specified parameter (from which the corner frequency and high-frequency spectral level are derived). Instead, the stress drop and ground motion levels at high frequencies are a function of the high-frequency ground motion levels embodied in the empirical source functions and the source duration of the finite rupture.

Comparisons of predicted and observed 5% damped response spectra are shown in Figures 3-9 to 3-12 for the four events modeled. Generally, the predicted spectra provide a good match to the observed spectra, especially at frequencies greater than 1 Hz. For the Loma Prieta earthquake, (Figure 3-9), there is a systematic underprediction of low frequencies, mainly below 1-2 Hz, but very good correspondence between predictions and observations at higher frequencies. For the Whittier Narrows event (Figure 3-10), which is a smaller event, deficient in low-frequency energy, there is a good match between predicted and observed spectra at all frequencies. While few data were available for the Nahanni event (Figure 3-11), the predicted spectra correspond well with the observed, but not quite as well as for the Loma Prieta and Whittier Narrows events. For the Saguenay event, not much energy is present at low frequency (Figure 3-12), and the model generally underpredicts the data above 1 Hz. This underprediction of high frequencies is probably due to the complexity and richness of high frequency radiation from this event.

3.3.3 Modeling Variability and Bias

The residuals between the recorded and simulated response spectra are used to estimate the variability associated with the simulation procedure. The bias and standard error, averaged over all stations for all events, are shown in Figure 3-13. Averaged over the four events, there is no significant bias (Figure 3-13a) for frequencies above 2 Hz. The standard error is about 0.5 for peak acceleration and for frequencies above about 10 Hz, and grows to about 0.6 at 3 Hz. Since the bias is low for frequencies above 2 Hz, the standard error (Figure 3-13b) does not change very much when corrected for model bias.

In this application of the semi-empirical ground motion model, the simulations

were done for a reference site condition appropriate for average surface conditions in the region in which each earthquake occurred. However, the data against which these simulations is compared used all available digital recordings regardless of site conditions. Consequently, the standard error of the modeling method contains a significant contribution from these unmodeled site effects. The incorporation of site effects in the semi-empirical procedure, as intended in these guidelines and implemented in the stochastic approach, would appreciably reduce the standard error.

3.4 Comparison of Models

As is evident from Sections 3.2 and 3.3, both the stochastic and semi-empirical ground motion models accurately predict ground motions for earthquakes of moderate magnitudes (M 5.8 to 7.0) over a broad range of frequencies, particularly above 1 Hz. It is worthwhile to reexamine the variability and biases of the two modeling approaches using only common event-station pairs. Altogether there are 39 sites for which recordings of horizontal acceleration spectra were modeled using both ground motion models. These stations are noted in Tables 3-2 to 3-5. The selection of additional sites chosen for each model comparison is discussed in Sections 3.2 and 3.3. The sites in common represent nearly all rock sites within 200 km of their respective sources. In particular, for the Whittier Narrows earthquake, a number of soil sites included in the semi-empirical validation were excluded from the stochastic model validation. Also, for the Saguenay earthquake, the stochastic model validation included a number of rock sites at distances beyond 200 km not considered for the semi-empirical model validation.

For the 39-station subset of sites, the standard errors of the residuals (modeling variability) and the modeling biases are presented for the two models in Figures 3-14 and 3-15. For the stochastic model, the differences in modeling bias between the predictions using the original 61-station set (Figure 3-6a) and the 39-station subset (Figure 3-14a) are not statistically significant. For the modeling variability, the total randomness and uncertainty for the 39-station set (Figure 3-14b) is actually lower than for the 61-station set (Figure 3-14b). This is largely due to a large contribution to the variability in the 61-station set from the Saguenay event; and many of these stations are not in the 39-station set.

For the semi-empirical model, the differences in modeling bias between the 104-station and 39-station predictions are not statistically significant. For the modeling variability, neither the differences in the random (corrected for model bias) nor the random and uncertain (not corrected for bias) portions of variability are much different for the two sets of stations. It is clear from this comparison that differences in station selection between the two validation procedures did not bias the results.

Although the ground motion models differ in certain fundamental ways regarding assumptions for source, path and site effects, both models yield accurate predictions of recorded strong motions. The stochastic model used does not incorporate finite source effects, and therefore is expected to predict motions less well at very close source distances. Generally speaking, however, this model has been shown to predict high frequencies with considerable accuracy, while overpredicting low frequencies at close distances. This effect is indicated in the response spectra in Figure 3-2 for distances < 50 km, and is shown by negative model bias in Figures 3-6a and 3-14a. In fact, these figures show that the stochastic model is conservative at all frequencies, significantly so below 3 Hz and marginally so at higher frequencies. At high frequency, particularly above 5 Hz, the incorporation of the site-specific kappa parameter provides a significant reduction (about 20%) in modeling variability.

The semi-empirical model uses a hybrid empirical-theoretical approach to incorporate finite source effects. This approach can provide a reduction in uncertainty for ground motion predictions for large earthquakes at very close distances. The lack of a site term tends to make this model have slightly higher uncertainty at higher frequencies, where site factors often dominate. Although the generalized ray method of modeling wave propagation effects used in this model (see Appendix 3.B) is expected to be slightly more accurate, especially at low frequencies (for a specific structure) than that used in the stochastic model (see Appendix 3.A), these differences do not appear to be significant in modeling the variability for the four events. These results indicate that both models produce comparably accurate predictions of strong ground motions over the frequency range of interest, 1 to 35 Hz.

The stochastic and semi-empirical ground motion models are each used in future sections, but for different purposes. For ground motion prediction of a future event,

the variability of source, path and site parameters causes extensive averaging of ground motion effects. These effects tend to smooth over the deterministic aspects of any given earthquake, and make the stochastic model particularly well suited to the task. The more computationally intensive and rigorous semi-empirical model, using generalized rays (Appendix 3.B), was used to develop the crustal structure regionalization in Section 5.

For the prediction of ground motion at rock sites in eastern North America, the results of Sections 3.2 and 3.3 indicate that both the semi-empirical and stochastic ground motion models are well validated. Due largely to its simplicity and computational efficiency, the stochastic model was selected to make the many thousands of simulations necessary to develop the engineering ground motion model described in Section 9. The results presented in this section indicate that both models would indeed have yielded quite similar results, both in the median and the variability. Thus, there is greater confidence that the results are not biased by implementing but one modeling method in the final analysis.

3.5 References

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Table 3-1

MODEL PARAMETERS AND DISTRIBUTIONS
FOR THE STOCHASTIC GROUND MOTION MODEL

<u>Parameter</u>	<u>Median</u>	<u>Sigma</u>	<u>Distribution</u>
$\Delta\sigma$	120 bars	0.70	log-normal
κ	0.006 sec	0.40	log-normal

Q(f) MODELS

<u>Midcontinent</u>	<u>Q₀</u>	<u>η</u>
mean	670	0.33
upper-range	1000	0.30
lower-range	400	0.40
<u>Gulf Coast</u>		
mean	300	0.30
upper-range	500	0.30
lower-range	200	0.30

Table 3-2

MODEL PARAMETERS FOR VARIABILITY CALCULATIONS
FOR THE STOCHASTIC GROUND MOTION MODEL

Earthquake	M	$\Delta\sigma$ (bars)	Q_0	η	κ (sec)	σ_{ln}	$e^{\sigma^2/2}$	N
Loma Prieta	6.9*	171	150*	0.60*	0.052	0.35	1.18	23
Whittier Narrows	5.9	165	150*	0.60*	0.038	0.43	1.15	12
Nahanni	6.8*	86	670*	0.33*	0.011	0.60	1.12	3
Saguenay	5.8*	655	361	0.87	0.026	0.87	1.19	20

*Fixed

Table 3.3
LOMA PRIETA MODELING SUMMARY

Name	Label	$\kappa(\text{sec})^{**}$	D	R(km)
Corralitos	COR [‡]	0.067	0.79	12.0
Los Gatos	LGPC [‡]	0.046	1.04	12.4
Branciforte Dr.	BRAN [‡]	0.032	0.87	15.6
Gilroy 1	GL1 [‡]	0.038	0.80	19.2
Gilroy 2	GL2 ^{*‡}	0.052	1.00	20.0
Santa Cruz	UCS [‡]	0.032	0.82	20.0
UCSC-Seismic Lab	UCSC [‡]	0.025	0.71	23.3
Gilroy 6	GL6 [‡]	0.060	0.77	26.8
Palo Alto VA	PAV ^{*‡}	0.052	1.00	26.8
SLAC	SLA [‡]	0.069	1.26	30.5
Redwood City-Canãda	RDC	0.065	0.63	39.9
SAGO South	SAS [‡]	0.080	0.76	40.8
APEEL 7	A07	0.065	1.10	45.6
APEEL 10	A10	0.087	1.18	45.6
Monterrey City Hall	MON [‡]	0.042	0.47	45.6
Belmont-2 story	BEL	0.072	1.01	47.5
APEEL 2E, CUSH	A2E [‡]	0.059	0.75	54.3
Sierra Point	SSP [‡]	0.036	0.93	66.1
SF Diamond Height	SFD [‡]	0.052	1.16	74.0
Piedmont J.H.	PHS [‡]	0.051	0.98	75.0
SF Rincon	SFR [‡]	0.029	1.04	76.9
Yerba Buena Island	YBI [‡]	0.073	1.03	77.9
SF Pacific Heights	SFH [‡]	0.069	1.14	78.9
SF Telegraph Hill	SFT [‡]	0.059	1.00	78.9
Treasure Island	TRI [*]	0.052	1.00	79.9
SF Cliff House	SFC [‡]	0.065	1.31	80.9

*Soil sites.

**Kappa values are relative to the fixed Q(f) of 150 f^{0.6}. For distances exceeding about 50 km, the kappa values are strongly coupled to the Q(f) model.

‡Also modeled using semi-empirical method, see Sections 3.3 and 3.4.

Table 3.4
WHITTIER NARROWS MODELING SUMMARY

Name	Label	$\kappa(\text{sec})$	D	R(km)
Garvey Reservoir-abut.	GVR [‡]	0.035	1.08	15.6
Whittier Narrows Dam-upst.	WND [‡]	0.026	0.54	15.9
Alhambra- 900 S. Fremont	ALH [‡]	0.044	0.84	16.8
San Marino	SNM [‡]	0.048	0.53	17.1
LA-Obregon Park	OBG [‡]	0.039	0.96	18.1
Whittier-Bright	WHB [‡]	0.039	0.87	18.3
LA-Bulk Mail	LBM [‡]	0.047	1.11	18.9
Vernon CMD	VRN [*]	0.069	1.26	20.3
Norwalk-Imperial South	NWK [*]	0.048	0.68	21.9
Downey-Co. Maint.	DOW [‡]	0.061	1.09	23.1
Mt. Wilson	MTW [‡]	0.012	0.29	23.8
LA Hollywood Storage Lot	HSL [‡]	0.029	0.58	28.8

*Soil sites.

[‡]Also modeled using semi-empirical method, see Sections 3.3 and 3.4.

Table 3.5
NAHANNI MODELING SUMMARY

Name	$\kappa(\text{sec})$	D	R(km)
S2 [‡]	0.012	0.61	9.2
S1 [‡]	0.021	1.34	9.6
S3 [‡]	0.005	0.61	21.1

[‡]Also modeled using semi-empirical method, see Sections 3.3 and 3.4.

Table 3.6
SAGUENAY MODELING SUMMARY

Name	$\kappa(\text{sec})^{**}$	D ^{***}	R(km)
S16 [†]	0.006	1.0	52.0
S17 [†]	0.003	1.0	69.9
S20 [†]	0.002	1.0	94.9
S08 [†]	0.010	1.0	97.4
A54	0.025	1.0	97.9
A61	0.022	1.0	98.6
A64	0.024	1.0	105.5
A16	0.029	1.0	117.4
S10 [†]	0.020	1.0	118.0
A21	0.025	1.0	124.2
A11	0.038	1.0	125.6
EBN [*]	0.035	1.0	231.3
SBQ [*]	0.038	1.0	311.0
GSQ [*]	0.045	1.0	315.0
TRQ [*]	0.015	1.0	332.5
MNT [*]	0.031	1.0	346.4
KLN [*]	0.059	1.0	390.5
GRQ [*]	0.025	1.0	392.2
OTT [*]	0.036	1.0	460.8
WBO [*]	0.032	1.0	468.3

* ECTN vertical component, H/V of 1.4 used to convert vertical records to horizontal.

**Kappa values for the horizontal component ECTN sites ANN may not be reliable due to the limited bandwidth. The vertical component sites also have a limited bandwidth and neglect any frequency dependence in the H/V ratio.

***Fixed.

†Also modeled using semi-empirical method, see Sections 3.3 and 3.4.

Table 3-7

SOURCE PARAMETERS OF EARTHQUAKES USED IN VALIDATION
OF SEMI-EMPIRICAL SIMULATION METHOD

Earthquake	Seismic Moment ($\times 10^{25}$ dyne cm)	Depth to Top Center (km)	Fault Length (km)	Fault Width (km)	Strike	Dip	Rake	Rupture Velocity (km/sec)	Rise Time (sec)
1985 Nahanni	17.6	3.56	32	15	180	22	110	2.7	0.96
1987 Whittier Narrows	1.0	12.35	12	9	280	30	110	2.5	0.4
1988 Saguenay	0.5	23.14	4	6	323	65	78	3.0	0.4
1989 Loma Prieta	29.5	3.38	40	18	128	70	140	2.7	1.14

Table 3-8

LOMA PRIETA EARTHQUAKE:
SEISMIC STATIONS USED IN VALIDATION OF SEMI-EMPIRICAL MODEL

CODE	NUMBER	ORG	STATION NAME	LAT	LONG	CLOSEST DISTANCE
agnw	57066	CSMIP	Agnews State Hospital	37.397	-121.952	24.33
andd	1652	USGS	Anderson Dam-Downst (DMD)	37.1642	-121.6314	19.64
andl	1652	USGS	Anderson Dam-Left Abut	37.166	-121.628	19.98
bran		UCSC	Branciforte Drive	37.047	-121.985	10.80
capi	47125	CSMIP	Capitola	36.974	-121.952	15.11
cidd	57504	CSMIP	Coyote Lake Dam-Down Strm	37.124	-121.551	20.53
cor	57007	CSMIP	Corralitos	37.046	-121.803	3.40
csuh	58219	CSMIP	CSUH- Stadium (see ap3e)	37.657	-122.061	52.37
frem	1686	USGS	Fremont-Emerson Court	37.535	-121.929	39.61
gav	47006	CSMIP	Gavilan Coll Phys Science	36.973	-121.568	9.50
gil1	47379	CSMIP	Gilroy #1-Gavilan Water T.	36.973	-121.572	9.17
gil2	47380	CSMIP	Gilroy #2-Hwy101/Bolsa Rd.	36.982	-121.556	10.63
gil3	47381	CSMIP	Gilroy #3-Sewage Plant	36.987	-121.536	12.40
gil4	57382	CSMIP	Gilroy #4-San Ysidro Schl	37.005	-121.522	14.08
gil6	57383	CSMIP	Gilroy #6-San Ysidro	37.026	-121.484	17.97
gil7	57425	CSMIP	Gilroy #7-Mantelli Ranch	37.033	-121.434	22.35
gilb	57476	CSMIP	Gilroy Historic Bldg (FH)	37.009	-121.569	10.56
hall	57191	CSMIP	Hall's Valley, Grant Park	37.338	-121.714	30.19
hol	47524	CSMIP	Hollister-South & Pine	36.848	-121.397	27.58
holc	1575	USGS	Hollister-City Hall Annex	36.851	-121.402	27.03
lbl	58471	CSMIP	Berkeley-Lawrence Berk Lab	37.876	-122.249	79.12
lex1	57180	CSMIP	Lexington Dam-left abutmnt	37.202	-121.949	6.25
lgpc		UCSC	Los Gatos Presenta Center	37.172	-122.010	3.45
lick	58135	CSMIP	UCSC-Lick Obs Elect Lab	37.001	-122.060	18.45
mont	47377	CSMIP	Monterey City Hall	36.597	-121.897	44.20
mpva	1230	USGS	Menlo Park VA Hospital	37.468	-122.157	33.57
mssj	57064	CSMIP	Fremont, Mission San Jose	37.530	-121.919	39.27
muir	58393	CSMIP	Hayward-Muir School	37.657	-122.0820	52.53
oako	58224	CSMIP	Oakland-2 story ofc bldg	37.806	-122.267	72.11

Table 3-8 (continued)

LOMA PRIETA EARTHQUAKE:
SEISMIC STATIONS USED IN VALIDATION OF SEMI-EMPIRICAL MODEL

CODE	NUMBER	ORG	STATION NAME	LAT	LONG	CLOSEST DISTANCE
olem	68003	CSMIP	Olema, Point Reyes	38.043	-122.797	117.14
pavb	1227	USGS	Palo Alto VA Hosp-Bsmnt	37.400	-122.140	26.08
pijh	58338	CSMIP	Piedmont Jr HS Grounds	37.823	-122.233	73.09
pres	58222	CSMIP	SF - Presidio	37.792	-122.457	77.46
ptba	58043	CSMIP	Point Bonita	37.820	-122.520	82.99
pulg	58378	CSMIP	Pulgas-Upper Crys Sprgs R	37.49	-122.31	42.35
rdwd	58263	CSMIP	Redwood City school ofc b	37.448	-122.265	36.26
rich	58505	CSMIP	Richmond-City Hall Pking	37.935	-122.342	87.76
sago	47189	CSMIP	SAGO - Hollister	36.753	-121.396	33.92
sali	47179	CSMIP	Salinas	36.671	-121.642	32.48
sfch	58132	CSMIP	SF - Cliff House	37.780	-122.510	78.78
sfdh	58130	CSMIP	SF - Diamond Heights	37.740	-122.430	71.29
sfgg	1678	USGS	SF - Golden Gate Bridge	37.806	-122.472	79.46
sfph	58131	CSMIP	SF - Pacific Heights	37.790	-122.430	76.11
sfrh	58151	CSMIP	SF - Rincon Hill	37.790	-122.390	74.50
sfth	58133	CSMIP	SF - Telegraph Hill	37.800	-122.410	76.28
sftt	1239	USGS	SF - Transamerica Title	37.80	-122.40	75.88
skyl	58373	CSMIP	Skyline-Upper Crys Sprgs R	37.465	-122.343	42.12
slac	1601	USGS	Stanford-SLAC Test Lab	37.419	-122.205	30.63
snj	57563	CSMIP	San Jose-Santa Teresa Hill	37.210	-121.803	14.35
srtg	58065	CSMIP	Saratoga-Aloha Avenue	37.255	-122.031	8.29
ssfs	58539	CSMIP	So SF - Sierra Point	37.674	-122.3880	63.11
sunc	1695	USGS	Sunnyvale-Colton Ave	37.402	-122.024	24.09
waho		UCSC	Santa Cruz, Walter's House	36.970	-121.996	17.71
wats	47459	CSMIP	Watsonville	36.909	-121.756	10.34
yerb	58163	CSMIP	Yerba Buena Island	37.810	-122.360	75.41

Table 3-9

**NAHANNI EARTHQUAKE:
SEISMIC STATIONS USED IN VALIDATION OF SEMI-EMPIRICAL MODEL**

CODE	NUMBER	ORG	STATION NAME	LAT	LONG	DISTANCE
Sta1			Station 1	62.20	-124.37	7.99
Sta2			Station 2	62.23	-124.18	3.98
Sta3			Station 3	62.12	-123.84	16.28

Table 3-10

WHITTIER NARROWS EARTHQUAKE:
SEISMIC STATIONS USED IN VALIDATION OF SEMI-EMPIRICAL MODEL

CODES	NUMBER	ORG	STATION NAME	LAT	LONG	CLOSEST DISTANCE
alhc	24461	CSMIP	Alhambra,Fremont School	34.070	-118.150	13.35
alhu	482	USGS	Alhambra,900 SFremont.bsmt	34.090	-118.150	14.34
alta	24402	CSMIP	Altadena: Eaton Canyon	34.177	-118.096	19.47
bulk	5129	USGS	LA: Bulk Mail Facility	33.990	-118.160	12.95
cast	24278	CSMIP	Castaic: Old Ridge Route	34.564	-118.642	71.90
down	14368	CSMIP	Downey-County Maint Bldg	33.924	-118.167	16.68
dume	24396	CSMIP	Malibu: Pt.Dume School	34.077	-118.800	61.44
feat	13122	CSMIP	Featherly Park Maint Bldg	33.869	-117.709	35.02
garv	709	USGS	Garvey Reservoir	34.050	-118.110	12.83
hadb	14395	CSMIP	Long Beach: Harbor Ad FF	33.754	-118.200	32.72
hasl	24277	CSMIP	Castaic: Hasley Canyon	34.459	-118.650	64.52
hbfs	13197	CSMIP	Huntington Beach: Fire Sta	33.662	-117.997	40.13
hemt	12331	CSMIP	Hemet: Stetson Ave Fire St	34.729	-116.979	120.48
hsbf	24303	CSMIP	LA: Hollywood Storage FF	34.090	-118.339	22.49
ingl	14196	CSMIP	Inglewood-Union Oil Yard	33.905	-118.279	21.61
kagc	24088	CSMIP	Pacoima: Kagel Canyon	34.288	-118.375	35.10
lacn	24389	CSMIP	Century Cty: LA CntryClubN	34.063	-118.418	27.88
lanc	24526	CSMIP	Lancaster: Med.OfcBldg	34.688	-118.156	68.08
lbrp	14241	CSMIP	Long Beach-Recreation Park	33.778	-118.133	29.34
leo5	24055	CSMIP	Leona Valley #5	34.600	-118.241	59.44
leo6	24309	CSMIP	Leona Valley #6	34.604	-118.244	59.90
moor	24283	CSMIP	Moorpark,Ventura Co. Fire	34.288	-118.881	73.87
mtwi	24399	CSMIP	Mt Wilson-CIT Seismic Sta	34.224	-118.057	22.91
newh	24279	CSMIP	Newhall: LA Cnty Fire Sta	34.390	-118.530	51.75
nord	24087	CSMIP	Arleta: Nordhoff Fire Sta	34.236	-118.439	36.20
obre	24400	CSMIP	LA: Obregon Park	34.037	-118.178	12.67
pomo	23525	CSMIP	Pomona - 4th & Locust FF	34.056	-117.748	28.71

Table 3-10 (continued)

**WHITTIER NARROWS EARTHQUAKE:
SEISMIC STATIONS USED IN VALIDATION OF SEMI-EMPIRICAL MODEL**

CODES	NUMBER	ORG	STATION NAME	LAT	LONG	CLOSEST DISTANCE
rair	13123	CSMIP	Riverside Airport	33.951	-117.446	55.09
ranc	23497	CSMIP	Rancho Cucamonga L&J FF	34.104	-117.574	43.62
rosa	24274	CSMIP	Rosamond, Goode Ranch	34.827	-118.265	83.92
schl	14403	CSMIP	LA: 116th St.School	33.929	-118.260	19.11
snmr	24401	CSMIP	San Marion: SW Academy	34.115	-118.130	15.82
sylm	24514	CSMIP	Sylmar: Olive View FF	34.326	-118.444	41.85
tarz	24436	CSMIP	Tarzana: Cedar Hill Nurs.	34.160	-118.534	40.30
vasq	24047	CSMIP	Vasquez Rocks Park	34.490	-118.320	50.03
wdup	289	USGS	Whittier Narrows Dam-Upst	34.030	-118.050	12.55
whit	804	USGS	Whittier: 7215 Bright Ave	33.977	-118.036	12.79

Table 3-11

**SAGUENAY EARTHQUAKE:
SEISMIC STATIONS USED IN VALIDATION OF SEMI-EMPIRICAL MODEL**

CODES	NUMBER	ORG	STATION NAME	LAT	LONG	CLOSEST DISTANCE
SM01			St.-Ferreol, Quebec	47.1256	70.8282	114.33
SM02			Quebec, Quebec	46.7782	71.2749	149.57
SM05			Tadoussac, Quebec	48.1432	69.7189	110.10
SM08*			La Malbaie, Quebec	47.6553	70.1527	94.01
SM09			St-Pascal, Quebec	47.5257	69.8045	123.03
SM10*			Riviere-Quelle, Quebec	47.4757	69.8045	114.82
SM16*			Chicoutimi-Nord, Quebec	48.4902	71.0123	48.02
SM17*			St-Andre-du-Lac-St-Jean	48.3248	71.9917	66.09
SM20*			Les Eboulements, Quebec	47.5496	70.3273	91.40

* Common to stations modeled by stochastic method.

Figure 3-1. Schematic of the stochastic ground motion model. To include crustal propagation effects the $1/R$ in the path operator is replaced by a geometrical attenuation term which includes the effects of direct and both sub- and post-critical reflected phases (Appendix 3A).

Figure 3-2. Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the stochastic ground motion model. (Solid line—average of two horizontal spectra; Dashed line—model.)

Figure 3-2. (*continued*). Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the stochastic ground motion model. (Solid line—average of two horizontal spectra; Dashed line--model.)

Figure 3-2. (continued). Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the stochastic ground motion model. (Solid line—average of two horizontal spectra; Dashed line—model.)

Figure 3-3. Comparison of observed and modeled 5% damped acceleration response spectra for the 1987 Whittier Narrows earthquake. Model results are from the stochastic ground motion model. (Solid line—average of two horizontal spectra; Dashed line—model.)

Figure 3-4. Comparison of observed and modeled 5% damped acceleration response spectra for the 1985 Nahanni earthquake. Model results are from the stochastic ground motion model. (Solid line—average of two horizontal spectra; Dashed line—model.)

Figure 3-5. Comparison of observed and modeled 5% damped acceleration response spectra for the 1988 Saguenay earthquake. Model results are from the stochastic ground motion model. (Solid line—average of two horizontal spectra or vertical spectrum : 1.4. Dashed line--model.)

Figure 3-5 (continued). Comparison of observed and modeled 5% damped acceleration response spectra for the 1988 Saguenay earthquake. Model results are from the stochastic ground motion model. (Solid line—average of two horizontal spectra or vertical spectrum : 1.4. Dashed line— - -model.)

Figure 3-6a. Modeling bias computed from recordings at 61 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the stochastic ground motion model. Dashed lines represent 90% confidence limits.

Figure 3-6b. Standard error of modeling or modeling variability (natural log) computed from recordings at 61 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the stochastic ground motion model. Solid line, total variability (modeling uncertainty plus randomness); dashed line, corrected for model bias and site term D.

Figure 3-7. Schematic diagram of the ground motion simulation procedure. Source: Somerville et al., 1990.

Figure 3-8. Schematic diagram of the wave propagation model, with direct waves (S) and waves $S_c S$ and $S_m S$ reflected from the Conrad and Moho layers. At the critical angle (i_{crit}), the incident wave is totally reflected back to the surface at distances starting at the critical distance R_{crit} .

Figure 3-9. Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-9 (continued)). Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-9 (continued)). Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-9 (continued)). Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-9 (continued)). Comparison of observed and modeled 5% damped acceleration response spectra for the 1989 Loma Prieta earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-10. Comparison of observed and modeled 5% damped acceleration response spectra for the 1987 Whittier Narrows earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-10 (continued). Comparison of observed and modeled 5% damped acceleration response spectra for the 1987 Whittier Narrows earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-10 (continued). Comparison of observed and modeled 5% damped acceleration response spectra for the 1987 Whittier Narrows earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-10 (*continued*). Comparison of observed and modeled 5% damped acceleration response spectra for the 1987 Whittier Narrows earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-11. Comparison of observed and modeled 5% damped acceleration response spectra for the 1985 Nahanni earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-12. Comparison of observed and modeled 5% damped acceleration response spectra for the 1988 Saguenay earthquake. Model results are from the semi-empirical ground motion model.

Figure 3-13a. Modeling bias computed from recordings at 104 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the semi-empirical ground motion model. Dashed lines represent 90% confidence limits.

Figure 3-13b. Standard error of modeling or modeling variability (natural log) computed from recordings at 104 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the semi-empirical ground motion model. Dashed line, total variability (modeling uncertainty plus randomness); solid line, corrected for model bias.

Figure 3-14a. Modeling bias computed from recordings at the sunset of 39 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the stochastic ground motion model. Dashed lines represent 90% confidence limits.

Figure 3-14b. Modeling variability (natural log) computed from recordings at the subset of 39 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the stochastic ground motion model. Dashed line, total variability (modeling uncertainty plus randomness); solid line, corrected for model bias and site term D.

Figure 3-15a. Modeling bias computed from recordings at subset of 39 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the semi-empirical ground motion model. Dashed lines represent 90% confidence limits.

Figure 3-15b. Modeling variability (natural log) computed from recordings for the subset of 39 stations for the Loma Prieta, Whittier Narrows, Nahanni, and Saguenay earthquakes using the semi-empirical ground motion model. Dashed line, total variability (modeling uncertainty plus randomness); solid line, corrected for model bias.

SECTION 4
QUANTIFICATION OF SEISMIC SOURCE EFFECTS

4

QUANTIFICATION OF SEISMIC SOURCE EFFECTS

There are three main parameters of the seismic source for the stochastic ground motion simulation procedure described in Section 3.2. These are magnitude, stress drop (corner frequency), and the relation between source distance and equivalent point-source distance (accounts for effect of extended sources). The development of the source parameters are briefly described in this section. A complete description of the development of the source parameters is given by EPRI (1993).

4.1 Magnitude

Both m_{Lg} and moment magnitude, M , are used to characterize earthquake size for the development of the engineering model in Section 9. The use of m_{Lg} provides a link to ENA earthquake catalogs and to current and past practice. The use of M provides a more direct and physically based estimate of ground motion. A relation between m_{Lg} and M including the effect of stress drop is developed in EPRI (1993) using a hybrid approach. The stochastic ground motion model in Section 3.2 is used to develop an average M_0 - m_{Lg} relationship. Empirical data are then used to validate this relationship and to quantify the variability in m_{Lg} given M_0 and the dependence of m_{Lg} on stress drop.

4.2 Stress Drop

For a fixed magnitude or moment, the stress drop in the stochastic source model (Eq. 3.1) directly scales ground motions for frequencies above the corner frequency (Silva, 1991). In the frequency range of interest, 1-35 Hz, the stress drop is therefore one of the controlling parameters for the point-source model (Section 9).

In EPRI (1993) two representations of stress drop are considered: the Brune stress drop and the high frequency stress parameter. While the differences in measured stress

drops using the two definitions are not significantly different, the high frequency stress parameter provides slightly more conservatism in estimated ground motion. Part of this conservatism may be due to the existence of a double corner in the source spectrum (Atkinson, 1993). The distribution of stress drop values is therefore constrained by values of the high-frequency stress parameter in developing the engineering model in Section 9. Stress drops ranging from 20 to 600 bars have been considered, and this variability is partitioned between randomness and uncertainty. The median stress drop for ENA is determined to be 120 bars.

4.3 Source Depth

The source depth distributions for ENA events developed in Section 5.2 are based on the locations of earthquake hypocenters (i.e., the point of initiation of source rupture). However, the stochastic ground motion model described in Section 3.2 yields simulations of ground motions relative to an asperity (i.e., a zone of greatest slip and greatest high-frequency radiation). Moreover, because the stochastic model used here considers source radiation from a point, geometrical effects of an extended fault that are significant at close distances are not explicitly considered in the simulations. Therefore, a conversion from hypocentral depth to asperity depth that accounts approximately for the extended source effect is derived. In addition, geometrical constraints on the spatial extent of the source are developed in terms of the probability that a large magnitude event will occur at shallow focal depths.

4.4 References

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- Silva, W. J. (1991). Global characteristics and site geometry, Proceedings: NSF/EPRI Workshop on Dynamic Soil Properties and Site Characterization, Electric Power Research Institute, Report NP-7337.

SECTION 5
QUANTIFICATION OF CRUSTAL PATH EFFECTS

QUANTIFICATION OF CRUSTAL PATH EFFECTS

Ground motion attenuation is the decay of ground motion amplitudes with increasing distance from the seismic source. Ground motion attenuation can vary across the U. S. due to differences in the properties of the rocks and Earth's crust. For example, as discussed in Section 1.5, the ground motions in the EUS attenuate less rapidly than ground motions in the WUS. These differences in the rate of attenuation of ground motion are called crustal path effects.

There are several factors that account for crustal path effects. These include large scale crustal velocity structure, anelastic absorption, and scattering. In the ground motion simulation procedures described in Section 3, path effects are included through 1-D velocity models and Q . The 1-D velocity model accounts for the large scale features of the crust such as the thickness of the crust (depth of the Moho boundary). The simplest models of crustal structure assume that the crust is a whole-space or a half-space. For a whole-space, the exponent of the distance term in Eq. 3.1, called the geometrical spreading coefficient, is unity (R^{-1} attenuation). The Q model accounts for anelastic absorption, scattering, and any other features that attenuate the ground motion differently from the geometrical spreading assumed in the model. For a given seismic source and focal depth, the geometrical spreading coefficient and Q account for the character of the ground motion attenuation.

Region specific crustal path effects can modify the rate of attenuation of ground motion such that the geometric spreading coefficient becomes distance dependent (see Appendix 3A). If the variations in crustal velocity structure and Q are large enough, the distance dependence of the geometrical spreading coefficient can be significantly different from region to region.

In this section, the objective is to determine if there are significant variations in the ground motion attenuation in the EUS that would warrant sub-dividing the EUS into multiple distinct ground motion attenuation regions. The approach used in this section is to first divide the EUS into multiple regions based upon seismological and geological factors. Numerical simulation procedures (Section 3.3) are then used to compute ground motions for each region. The simulated ground motions for each region are then compared to one another to determine if the seismological and/or geological differences result in significantly different ground motion attenuation. Regions that produce similar ground motion attenuation are combined into groups of ground motion attenuation regions. In Section 9, separate attenuation relations are derived for each distinct ground motion attenuation group.

5.1 Importance of Consideration of Both Velocity Structure and Q

Differences in observed ground motion attenuation rates are often attributed to differences in Q ; however, the velocity structure can also have an important effect on ground motion attenuation. The importance of considering effects of both Q and velocity structure variations is discussed below.

The influence of Q on the attenuation of ground motion grows with the length of the path, and is frequency dependent (see Eq. 3.1). Within one hundred km of the source, its influence in the eastern United States is relatively small except in the Gulf Coast region. However, for the frequencies of interest here (above 1 Hz), it is expected to become increasingly important beyond 100 km, and beyond a few hundred km the rate of decay of ground motion attenuation is expected to be dominated by Q effects.

In contrast, velocity structure effects are most important for source distances less than two hundred km. At short distances, the attenuation is affected by the velocity gradient between the seismic source and the receiver. At moderate distances, reflected waves can influence the rate of attenuation. This effect is illustrated schematically in Figure 5-1, in which the effect of both the direct wave (labelled S) and waves that are reflected from discontinuities in seismic velocity and density such as the Conrad and Moho layers (labelled S_cS and S_mS) are included. The effect of crustal structure can become significant when the reflected waves reach the critical angle, i_{crit} , and are completely reflected back to the surface at the critical distance, R_{crit} . In the critical distance range, the velocity structure can lead to distance dependent variations in the rate of

attenuation (increase or decrease of geometrical spreading coefficient). Beyond two hundred km, the large number of multiply reflected phases contributing to the ground motions, and the larger distance over which scattering effects can occur, subdues effects of velocity structure on the rate of attenuation.

Since the effects of velocity structure and Q both contribute to the shape of the attenuation curve, there are tradeoffs involved in the estimation of each from a set of empirical data. Since the effect of Q is relatively small inside 100 km, this tradeoff should not result in a great deal of uncertainty in the major effects of velocity structure at these distances, such as the effect of critical reflections from the lower crust. However, at distances beyond 100 km, the effects of velocity structure and Q become more difficult to separate given that Q represents all factors that affect the attenuation that are not included in the geometrical spreading model. In this situation, it is usually necessary to make certain assumptions about the form of the Q function or the geometrical spreading coefficient at large distances in order to develop a set of consistent parameters describing these two effects (Atkinson and Mereu, 1992).

5.1.1 Demonstration with Empirical Attenuation Data

Atkinson and Mereu (1992) derived an empirical ground motion attenuation function from a set of ECTN recordings of one hundred small and moderate magnitude earthquakes in southeastern Canada. As an example, the data at 2 Hz are shown in Figure 5-2a. Station corrections and event corrections have been applied to the data. The median curve from a regression using a trilinear form of the attenuation is shown by the solid line. Averaged over about one hundred events, the geometrical spreading term, assuming a large Q (no effect of Q), was found to be $R^{-1.1}$ within 70 km and $R^{0.0}$ between 70 and 130 km; beyond 130 km the data are compatible with geometric spreading coefficients in the range of 0.5 to 1.0 depending on the assumed Q model. The attenuation for $1/R$ and for the Mid-continent structure computed using the stochastic model from Section 3.2 are also shown in Figure 5-2a. The curves for the stochastic model have been averaged over focal depths of 5, 10, and 15 km using focal depth weights that are consistent with the Atkinson data set (See Appendix 5b). In the distance range of 50 to 120 km, the Mid-continent structure model produces flattening of the attenuation in comparison to $1/R$ attenuation that is similar to the flattening for the empirical curve. For this data set, the use of the Mid-continent structure produces a better fit to the empirical attenuation than the use of $1/R$ attenuation over the distance

range of 50 to 100 km. The median regression values for other frequencies are also shown in Figure 5-2b. They also shown the flattening of the attenuation in the range of 70 to 130 km. This flattening of the attenuation function in the distance range of 70 to 130 km is consistent with crustal structure effects resulting from critical reflections from layers in the lower crust (Burger et al., 1987). In a separate study, the flat shape in the attenuation of strong ground motion from the 1988 Saguenay earthquake in the distance range of 60 to 120 km was attributed by Somerville et al. (1990) to the effect of critical reflections from the lower crust. The distance dependence of the geometric spreading coefficient shows that the effect of crustal velocity structure on ground motion attenuation can be important.

5.1.2 Demonstration by Modeling of Attenuation from Small Events

The effect of crustal velocity structure on ground motion attenuation can also be seen by modeling the attenuation from small magnitude events. The amplitudes of recorded ground motions of ENA earthquakes are compared to those calculated using synthetic seismograms that include the effects of wave propagation in a layered crust.

The simulation procedure used to generate synthetic seismograms is similar to that described in Section 3.3. The principal difference is that a single source function, constructed to match a specified source spectrum, is used in place of empirical source functions. Due to the small set of rays used in the calculations, the simulations are not appropriate for distances greater than 200 km.

The Fourier amplitude of the source function is taken as a Brune source with a stress drop of 100 bars. (Since we will be comparing the relative rates of attenuation between regions and not the absolute amplitudes, the value of the stress drop is not important.) The phase spectrum for each source has been derived from a segment of the SM16 recording of the 1988 Saguenay earthquake having a duration of the inverse of the lower corner frequency of the Saguenay event.

5.1.2.1 Analysis of Ten Grenville Events. Initially, we modeled the attenuation of ten earthquakes that were well recorded by the ECTN in the Northern Grenville-Superior Region of southeastern Canada. We generated source spectra by estimating the seismic moment from the Lg magnitude and using a Brune source spectrum with a stress drop of 100 bars. Since the seismic moment estimates derived in this way are subject to

considerable uncertainty, we normalized the synthetic seismograms to fit the recorded response spectral velocity at 5% damping at 2 Hz, averaged over stations within 200 km. Although we were not able to check absolute amplitude information with this procedure, we were able to compare the shapes of the observed and synthetic attenuation. Frequency-dependent station corrections derived by Atkinson and Mereu (1992) were applied to the recorded data.

The locations of the ten events, and of the recording stations, are shown in Figure 5-3, and the events are listed in Table 5-1. Five of the events occurred in the West Quebec seismic zone. For these events, we used a crustal structure model derived from the 1982 COCRUST experiment (Mereu et al., 1986), shown on the left side of Figure 5-4. The other five events occurred in or near the Charlevoix seismic zone. For these events, we used a crustal structure model derived by Woodward-Clyde Consultants (1991) from modeling the Saguenay earthquake sequence, shown on the right side of Figure 5-4.

The ten Grenville events include both shallow and deep earthquakes in both the West Quebec and Charlevoix regions. From an analysis of the empirical data and from modeling, we find that the largest variation in the attenuation among these events is due to differences in focal depth, and that the differences in crustal structure are of secondary importance. To illustrate this effect, we selected the events of 03/10/88 and 11/11/87, at depths of 12 and 21 km, respectively, in the West Quebec region, and the events of 90/04/21 and 88/11/25 (the Saguenay earthquake), at depths of 10 and 26 km, respectively, in the Charlevoix region. Profiles of recorded and synthetic vertical velocity seismograms are compared in Figures 5-5 through 5-8 for each of the four events. The records are aligned in absolute time, and the expected times of the direct and reflected waves are shown. Velocity waveforms are displayed to facilitate the correlation of seismic phases.

For the shallow West Quebec event of 03/10/88, the largest arrivals in the recorded seismograms occur near the travel time of the direct wave at distances within 70 km. However, at stations beyond 70 km, the largest arrivals in the data occur following the times of the Conrad and Moho reflections. In the synthetic seismograms, the largest arrivals occur with the direct wave at distances less than 100 km, and with the Moho reflection at distances larger than 100 km.

For the deep West Quebec event of 11/11/87, the phases contributing the peak velocity on the recorded seismograms show a clear progression from the direct wave at the stations within 50 km to the Conrad reflection at stations between 50 and 100 km and to the Moho reflection at stations beyond 150 km. In the synthetic seismograms, the Conrad does not produce the large arrivals seen in the data, but the Moho reflection does.

For the shallow Charlevoix event of 04/21/90, the largest velocities are associated with the direct wave at stations within 40 km, while at 140 km there are contributions from several different arrivals. A similar pattern is seen in the synthetic seismograms.

For the deep Saguenay event of 11/25/88, the phases contributing the peak velocity on the recorded seismograms progress from the direct wave at the two closest stations to the Conrad reflection for stations between 80 and 115 km to the Moho reflection at stations beyond 120 km. In the synthetic seismograms, the Moho reflection becomes strong at distances beyond 80 km.

The attenuation of recorded and synthetic response spectral velocity at 2 Hz for 5% damping for shallow and deep events are compared in Figure 5-9. The top panels include the three deep events (deeper than 20 km) and the bottom panels include the seven shallow events. For each set of events, the data in the distance range of 0 to 200 km are shown on the left and the synthetic values are shown on the right. The ground motion values for each event were normalized by fitting a least squares straight line and selecting a specific distance (40 km) at which the amplitudes were normalized. These lines are shown together with the ground motion values. The dashed lines show the $1/R$ attenuation for the average focal depth of each subset. For deep sources (top panels of Figure 5-9), the wave propagation model predicts a slower rate of attenuation than does the $1/R$ model. The data (top left panel) also show attenuation that is slower than $1/R$. For shallow sources (bottom panels), both the wave propagation and the data are consistent with $1/R$ attenuation except at short distances. The two data points at distances less than 10 km show larger ground motions than predicted by $1/R$ attenuation. The wave propagation model predicts a similar high motion for one of the two data points, but the model predicts the same as $1/R$ for the other data point.

There is a systematic difference in the slopes of the lines for the shallow and deep events; this difference is more pronounced in the data than in the synthetics. In both the

West Quebec and Charlevoix regions, deep events attenuate more gradually than shallow events in the distance range of 0 to 200 km. The attenuation of the shallow events is similar to the average attenuation function derived empirically from about one hundred events, described by Atkinson and Mereu (1992; Figure 5-2), reflecting the fact that most of these events had shallow focal depths. In summary, in both the West Quebec and Saguenay-Charlevoix regions we find that the shape of the attenuation in both the recorded and synthetic data is dependent on the focal depth of the earthquake.

The residuals for all ten events are plotted as a function of distance in Figure 5-10 for response spectral velocity at 1, 2, 5 and 10 Hz. Residuals for shallow events (shallower than 20 km) are shown as open boxes, while residuals for deep events are shown as crosses. Generally, for each of these depth categories, there are no significant trends with distance, indicating that the shape of the attenuation of the data is adequately matched by the wave propagation method. The overall lack of distance trends in the residuals indicates that the flattening of the attenuation curve due to critical reflections is consistent between the data and the wave propagation model. The residuals, averaged over distance, for frequencies of 1, 2, 5 and 10 Hz, are tabulated in Table 5-2. The response spectral standard errors are about $0.235 \log_{10}$ units (a factor of 1.7) on average.

We tested whether using the actual focal depth of the earthquake produced a better fit to the recorded data than assuming a focal depth of 11 km for all events. The residuals increased by about 10% when a focal depth of 11 km was used in place of the actual focal depth. This is a further indication that the focal depth influences the recorded data in the sense predicted by the wave propagation method.

5.1.2.2 Analysis of Ten Eastern North American Events. To evaluate how well ground motion attenuation inside 120 km in some other regions of eastern North America is modeled by our simulation procedure, we selected an additional six events that occurred in three other regions. These events occurred in the Atlantic Coastal Plain, Southern Appalachian, and New Madrid Regions, and are listed together with the selected Grenville events (identified by asterisks) in Table 5-1. The locations of the events are shown in Figure 5-11. We used the same procedure as for the Grenville events, except that station corrections were not available for the New Madrid event.

The attenuation for these events is described qualitatively as follows. The attenuation for the Giles County earthquake in the Southern Appalachian Region is gradual in the distance range of 30 to 100 km, due to critical reflections from the Conrad, is matched well by the model. Two earthquakes in Central Virginia in the Atlantic Coastal Plain Region have recordings close enough (20 km) to show the rapid attenuation at short distances before the critical reflection from the Conrad layer causes more gradual attenuation beginning at 50 km. Two earthquakes in eastern Tennessee in the Southern Appalachian Region occurred beneath the Conrad and have rapid attenuation out to 100 km. An earthquake in the New Madrid Region has fairly rapid attenuation out to 100 km, where critical Moho reflections begin to arrive and make the attenuation more gradual.

The residuals for all ten events are plotted as a function of distance in Figure 5-12 for response spectral velocity at 1, 2, 5 and 10 Hz and for peak acceleration (within the 12 Hz bandwidth of the instrument) and peak velocity. Residuals for shallow events (shallower than 20 km) are shown as open boxes, while residuals for deep events are shown as crosses. As for the ten Grenville events shown in Figure 5-10, there are generally no significant trends with distance for each depth category, indicating that the shape of the attenuation of the data is consistent with the wave propagation method. The residuals, averaged over distance, for frequencies of 1, 2, 5 and 10 Hz are tabulated in Table 5-2. The response spectral standard errors are about $0.206 \log_{10}$ units (a factor of 1.6) on average.

5.1.3 Conclusions

The simple wave propagation model that we have used generates arrivals due to the direct wave and to waves reflected from boundaries in a flat layered crustal model. For the sixteen eastern North America events that we have analyzed, we generally see that the timing of the largest arrivals in the empirical data is consistent with the timing of the arrival of reflected waves. The simulations which include crustal structure effects produce a better match to the data than simple $1/R$ attenuation, particularly for deep events. The overall lack of distance trends in the residuals indicates that the flattening of the attenuation curve due to critical reflections is similar in the data and in the wave propagation model. We conclude that crustal structure can have an important effect on ground motion attenuation for distances within 200 km.

5.2 Regionalization of Crustal Velocity Structure, Q, and Focal Depth

As shown in Section 5.1, the velocity structure and focal depth can significantly impact the propagation of seismic waves. In addition, Q also has a significant impact on ground motion attenuation. Variations of crustal velocity structure, Q, and focal depth distribution form the basis for regionalization of crustal path effects.

5.2.1 Crustal Velocity Structure Regionalization

A large body information on the crustal structure of the eastern United States has recently been summarized by Pakiser and Mooney (1989) in a volume entitled "Geophysical Framework of the Continental United States." From the information presented in this volume, it is evident that the velocity structure of the earth's crust in the central and eastern United States varies considerably from region to region. Differences exist in the thickness of the crust, the velocities of the upper and lower crust, and the presence or absence of surficial sedimentary deposits. Variations result primarily from differences in composition and tectonic history. By considering crustal structure in a broad sense, it is possible to identify large regions in which the average structure is similar. These regions can then form the basis for regional models of attenuation founded on calculations of wave propagation in these different crustal structures.

Although the velocity structure of the crust in the central and eastern United States is complex and three-dimensional, it is possible to describe separate regions based on average crustal velocity structure. In a previous study for EPRI (Woodward-Clyde Consultants, 1991), information on crustal structure was reviewed to identify 16 regions in which the structure is uniform to a first approximation. Crustal P-wave velocities for these regions were modeled as horizontal layers of uniform thickness. Some region boundaries are uncertain primarily from insufficient data coverage. Detailed seismic investigations of crustal velocity structure are usually carried out in areas of geologic interest or where anomalies are known to be present. These are not necessarily the best areas to resolve the placement of region boundaries or to characterize the average crustal structure between these boundaries.

In defining regions of similar crustal structure, three criteria played a major role: crustal thickness, the presence or absence of a high-velocity (P-wave velocity > 7 km/sec) lower crust, and the velocity structure of the upper crust. These factors were applied in

sequence to arrive at the final regionalization. The two cross sections across the central United States at latitudes 37° and 49°, developed by Braile (1989) and reproduced in Figure 5-13, were especially helpful in developing the regionalization and the velocity models of the regions. In addition, correlations with geologic or tectonic structures were used to guide the delineation of region boundaries. The interpretations of gross geologic structure for the northern and southern Appalachians (summarized in Taylor, 1989), and the crustal age terranes identified by Bickford and others (1986) were especially useful.

5.2.1.1 Crustal Thickness. Over much of the central and eastern United States, crustal thickness is fairly uniform and lies between about 35 and 45 km (e.g., Braile, 1989; Taylor, 1989; Mooney and Braile, 1989). Three regions exhibit a somewhat thicker crust of about 45 to 50 km: the south-central Appalachians from eastern Tennessee to northern Virginia, the Superior basin beneath Lake Superior, and the western-most Great Plains. Along the Atlantic and Gulf coasts, the crust thins to 30 km or less.

5.2.1.2 Presence of High Velocity Lower Crust . The presence of a high-velocity lower crust provides another basis for differentiating crustal regions in the central and eastern United States. This layer, if present, is characterized by a velocity of about 7.2 to 7.4 km/sec, and a thickness of 10 to 15 km. High-velocity lower crust appears to exist along much of the southern and western edge of the craton in the central United States (Braile, 1989). Braile (1989) interprets the existence of such a layer along latitude 37° N from the southern Rocky Mountains to the Appalachians. To the north, his summary of crustal structure shows a similar layer associated with the Williston basin (Braile, 1989). A profile in eastern Colorado, however, does not show this layer (Braile, 1989). A high-velocity lower crust also exists beneath central New England (Taylor, 1989). This layer probably represents mafic material associated with an island arc that was accreted to the North American craton during Ordovician time. Various geologic evidence suggests that the island arc was founded on oceanic material (Taylor, 1989). In contrast, accreted island arc material in the southern Appalachians is not associated with a high-velocity lower crust. In this region the island arc appears to have developed on transitional (rifted) continental crust (Taylor, 1989).

5.2.1.3 Upper Crustal Structure. The third factor considered in defining regions of similar crustal velocity is the structure of the upper crustal layers. In general, this structure is described by a layer of about velocity 6.0 km/sec overlying a layer of about

velocity 6.6 km/sec. In some areas such as the coastal margin of the Gulf Coast from the Mississippi Delta to East Texas, a lower velocity layer is observed near the surface associated with sedimentary deposits, as described in Section 5.5.5. In a region centered on central Tennessee, the velocity of the surface layer is anomalously high (6.4 km/sec). Off the coast of New England, the lower velocity upper crustal layer appears to be missing.

5.2.1.4 Crustal Structure Regions. The crustal velocity structure regions are listed in Table 5-3 and shown in Figure 5-14. Crustal velocity models for each region are given in Appendix 5A, together with a summary of the considerations that led to the selection of each crustal model. A summary plot of the P-wave velocity structures is shown in Figure 5-15. Generally, there is no parallel information on shear wave velocity structure, and we have assumed Poisson's ratio in the crust to be 0.25. This is compatible with the average value of 0.258, with a standard deviation of 0.034, derived from 64 estimates of crustal Poisson's ratio in continental North America by Braile et al. (1989). To estimate densities (ρ), we used the relation: $\rho = 0.162 \alpha + 1.723$, for P wave velocity α larger than 3.0, based on the data of C. Drake shown in Figure 7-7 by Grant and West (1965).

Parallel to the development of this regionalization of crustal structure, a regionalization of eastern North America based on genetic origin was developed independently (EPRI, 1991). This genetic regionalization is shown in Figure 5-16. As would be expected, there is some correspondence between the structural and genetic regions, although the boundaries are generally not identical and there is a larger number of genetic regions than structural regions. The approximate correspondence between the two regionalizations is indicated in Table 5-3. The genetic (tectonic) domains are used to regionalize focal depth in Section 5.2.3.

5.2.2 Q Regionalization

As mentioned in Section 5.1, the Q model depends on the selected geometrical spreading model. However, for the tests of regional differences in ground motion attenuation, we are concerned only with relative differences in the ground motion and not the absolute levels. Therefore, we only need to use Q models that are consistent with one another and not necessarily consistent with the geometrical spreading model.

In a previous study for EPRI, Gupta et al. (1989) developed a consistent regionalization of Q models for North America. Their data set consists of short-period, three-component recordings from the LRSM (Long Range Seismic Measurements) and RSTN (Regional Seismic Test Network) stations. A total of over 1300 recordings from 107 earthquakes at distances up to 2000 km were studied. The Lg phase was narrow-bandpass filtered to obtain spectral amplitudes over the frequency range of 0.5 to 8 Hz. Observed spectral amplitudes were decomposed into station, source, and distance terms by regression. They found that there is a substantial increase in Q with frequency of about a factor of 3 from 1 to 8 Hz.

The Q values were separated into 8 geophysically distinct regions based on Bouguer gravity contours that delineated major divisions of the ENA crust (EPRI, 1986). The 8 regions are shown in Figure 5-17. Although there is often a correlation between crustal velocity structure and Q, the Q region boundaries do not correspond to the velocity structure region boundaries shown in Figure 5-14. The Q regions were correlated to the velocity structure regions based on the extent of the overlap of the regions with additional consideration for the location of the majority of the seismicity in the velocity structure region. That is, more weight was given to areas of overlap that contained the majority of the seismicity in a velocity structure region. The correlation of the velocity structure and Q regions is listed in Table 5-4. Only 5 of the 8 Q regions are used as Q models for the 16 velocity structure regions. The Q values computed by Gupta et al. for the 5 regions are listed in Table 5-5.

The Q values from Gupta et al. were fit to the form

$$Q = Q_0 f^{\eta} \quad (5-1)$$

using a weighted least-squares regression with the weights inversely proportional to the variance of the observation. As shown in Figures 5-18a - e, the 1.25 Hz Q value deviates significantly from the trend of the frequency dependence at the higher frequencies implying a change in the functional form of the Q model at low frequencies. Since this study is primarily concerned with frequencies greater than 1 Hz and Q does not significantly affect ground motions calculations in the distance range of interest at frequencies below a few Hz, the 1.25 Hz values were not used in the regression analysis. Excluding the 1.25 Hz values leads to an under estimation of Q, hence a slight over-estimation of ground motion, at low frequencies. The resulting Q models are listed

in Table 5-6 and are plotted against the data in Figures 5-18a - e. The five Q models are compared in Figure 5-19. Four of the five Q models are similar. The exception is Q8 (Gulf Coast region) which has much lower Q than the other four models.

5.2.3 Focal Depth Distribution Regionalization

In addition to the crustal properties, the focal depth of earthquakes can also have a significant effect on ground motion attenuation. Systematic variations in focal depth between regions based on tectonic type is also used as a basis for regionalizing ground motion attenuation in the EUS.

The rate of seismicity in ENA is low and the catalog of moderate to large magnitude earthquake in ENA with known focal depth is sparse. Therefore, we consider earthquake in other analogous tectonic regions to provide a larger data set with greater statistical significance. ENA is classified as a Stable Continental Region (SCR). Other SCR regions include parts of Africa, Asia/Russia, Australia, Europe, India, and South America.

The focal depth distributions are developed using the EPRI SCR Earthquake Catalog (Johnston, 1993). All events in the EPRI data base of magnitude 5 or larger having an assigned tectonic type are considered. For events with multiple depth assignments, the preferred depth is used. Of the six SCR regions, only Africa and Australia had earthquake depth distributions similar to that of ENA. The depth distributions of these three continents and their combined depth distribution are shown in Figure 5-20. The combined distribution of Eastern North America, Africa, and Australia will be referred to as EAA.

The characteristics of earthquake focal depths in eastern North America have been summarized by Wheeler and Johnston (1993) as follows. The depths of these earthquakes range from 3 to 32 km. There may be systematic variations of focal depth with geological terrain. The deeper events may indicate the presence of mafic or granulite rocks, whereas unusually shallow large earthquakes have only been observed in the central craton. In the central craton, earthquake depths are concentrated at about 5 km, with a range of 3 to 18 km. In the New Madrid region, focal depths are concentrated at about 7 km with a range of 5 to 12 km. Appalachian earthquakes tend to be shallower (in the depth range of 5 to 8 km) than those of comparable size in the late

Proterozoic continental margin that underlies the borders of the northwest Appalachian orogen (in the depth range of 8 to 14 km). Deep earthquakes (> 25 km) have only been observed beneath the late Proterozoic margin.

Based on the apparent correlation between focal depth and geologic terrain, the depth distributions for the ENA and EAA are subdivided according to tectonic type: margin, rift, or other. Margins include all crust between the landward limit of normal faulting or significant tectonic thinning and the continental/oceanic transition zone. Rifts are zones of large-scale normal faulting embedded within continental crust and include all crust that lies within the outermost rift-related normal faults. Regions of stable continental crust that are neither margins nor rifts are classified as "other" regions. A map showing the rift, margin and "other" regions in ENA is shown in Figure 5-16. The depth distributions for EAA are shown in Figure 5-21. The depth distributions for both ENA and EAA are tabulated in Table 5-7.

The statistical properties of the depth distribution for margins, rifts, and other regions are compared in Appendix 5B for both the ENA and EAA data sets. These comparisons are summarized below. The depth distributions for rifts and "other" regions are not significantly different from each other so these two categories are combined into non-margins. The depth distribution for margins is significantly different from that of non-margins at the 95% confidence level. This comparison only deals with the focal depths and not with the resulting ground motions. The effect of the different focal depths on ground motion is addressed in Section 9.

In comparing the ENA and EAA distributions we found that the larger EAA data set is applicable to the ENA at all focal depths for margins and at focal depth less than 30 km for non-margins. Therefore, the EAA depth distribution for non-margins is truncated at 30 km for application to the ENA. The resulting depth distributions smoothed over 5 km distance bins are shown in Figure 5-22. The corresponding normalized depth weights for the two tectonic categories are listed in Table 5-8. A generic model that does not distinguish between margins and non-margins is listed in Table 5-9.

5.3 Effects of Crustal Velocity Structure on Ground Motions

The effects of crustal structure are studied using numerical simulations. For this analysis, the semi-empirical simulation procedure using generalized rays described in

Section 3.3 is used to generate ground motions for each crustal structure. In this section, we consider the effects of crustal structure variations between regions (inter-region variations) as well as crustal structure variation within a region (intra-region variations).

5.3.1 Intra-Regional Variation (For Fixed Q)

The intra-regional variations of ground motion attenuation are estimated for a fixed Q model. That is, only the effects of changing the velocity structure of the crust are considered. The resulting variability represents our uncertainty in determining the crustal velocity structure for a given region. Three different effects are considered: variations of the velocities and layer thicknesses of the flat layered model, variations of Poisson's ratio, and variations from a flat-layered model (laterally varying structure).

5.3.1.1 Effect of Uncertainty in Crustal Structure within a Region. Within a given region, there are several sources of uncertainty in the idealized crustal structure model used to model ground motion attenuation. First, there is uncertainty in the crustal structure at a given location within a region due to the limitations of experimental measurements. This may be reflected in different models of crustal structure derived from the same experimental data, or from different sets of experimental data recorded over the same profile. Second, there may be variation in the actual crustal structure within the region. This may be reflected by differences in crustal structure obtained in different locations within the region.

We used four different crustal models from the Northern Grenville - Superior region, and four different crustal models from the New Madrid Rift region to evaluate the effect of these uncertainties in crustal structure on the variability of ground motions. The same Q model was used in each structure to isolate the effects of the velocity structure. Synthetic seismograms were computed for source depths of 4.5, 9.5, 14.5, 19.5, 24.5, and 29.5 km. (The source depths are at fractional integer depths to avoid locating the source at a layer boundary.)

The four crustal models for the Northern Grenville - Superior region are listed in Table 5-10 and shown in Figure 5-23. The West Quebec, Charlevoix and Berry models represent models at different locations in the region, whereas the West Quebec and Gradient models represent different models at the same location.

The West Quebec model for the Central Metasedimentary Belt near Mont Laurier, Quebec was derived from the profile for Point C in Line CD in Mereu et al. (1986). The Gradient model for the Central Metasedimentary Belt was derived from Point C (the same point as in the West Quebec model) in the intersecting Line CB in the same study. The Gradient model is identical to the West Quebec model above 30 km, but below 30 km has a velocity gradient to a deeper Moho in place of a step change in velocity at the Moho. The Charlevoix model for the north shore of the St. Lawrence River in Quebec was derived by Somerville et al. (1990) using data from the Saguenay earthquake sequence of 1988. The Berry model for northeast Quebec was derived by Berry and Fuchs (1973).

The four crustal models for the New Madrid Rift region are listed in Table 5-11 and shown in Figure 5-24. The McCamy, Ginzburg, and Saikia models represent models at different locations in the region, while the McCamy and Flank models represent different models at the same location.

The McCamy model was derived by McCamy and Meyer (1966) from a line from Cape Girardeau, Mo. to Little Rock, Ark. that lies along the northwest flank of the rift. The Flank model was derived by Ginzburg et al. (1983) from a profile that overlies that of the McCamy model. The Ginzburg model, derived in the same study, is for a profile along the axis and through the center of the New Madrid rift. The Saikia model was derived from modeling of earthquake and explosion body waves and surface waves near Dyersburg, Tennessee on the southeast flank of the rift by Saikia et al. (1992).

The standard error of spectral acceleration is shown as a function of distance for the New Madrid region for 1, 5 and 15 Hz in Figures 5-25a,b,c. In these figures, the standard error is shown separately for source depths of 4.5, 9.5, and 19.5 km. The ground motions from shallow focal depths are more sensitive to the velocity structure than the ground motions from the deep focal depths. The standard error in ground motion amplitude has a maximum at distances of 30 to 80 km associated with the variation in the Moho depth. The standard error of spectral acceleration for the Grenville region for 1, 5 and 15 Hz is shown in Figures 5-26a,b,c. For this region, the maximum standard error is shifted to distances of 80 to 120 km again associated with the variation in Moho depth. The depth of the Moho affects the distance range at which

the maximum standard error occurs. In general, the deeper the Moho, the larger the distance at which the maximum occurs.

In Figures 5-25 and 5-26 the variation of ground motion for a given crustal structure is shown for three depths. To account for the variability in focal depth, the ground motion variability is computed using a weighted average over focal depths where the weights are given by the EAA generic depth distribution (Table 5-9). The standard errors of the mean spectral acceleration are shown in Figures 5-27a,b,c for 1, 5, and 15 Hz.

5.3.1.2 Uncertainty in Poisson's Ratio. The regionalization of crustal structure described in Section 5.1 is based of necessity on P-wave velocity structure, because S-wave velocity structure is typically not determined in crustal refraction studies. To estimate S-wave velocity models, which are more important than P-wave velocity models for strong ground motion modeling, we assumed a Poisson's ratio of 0.25 with a standard deviation of 0.034, derived from 64 estimates of crustal Poisson's ratio in continental North America by Braile et al. (1989). This uncertainty in the value of Poisson's ratio that is used to convert the P-wave velocities to S-wave velocities constitutes an additional source of uncertainty in crustal velocity structure. The uncertainty in ground motion attenuation due to this uncertainty in Poisson's ratio was estimated by perturbing the S-wave velocities of the Mid-continent region velocity model (see Section 5.2) by an amount corresponding to plus or minus one standard deviation in Poisson's ratio. The resulting crustal models are listed in Table 5-12.

Ground motions were computed for the Mid-continent crustal structure using the three Poisson's ratios. The variability of ground motion including the effect of uncertainty in Poisson's ratio is not significantly larger than the variability due to crustal structure uncertainty alone. Therefore, uncertainty in Poisson's ratio is not considered further.

5.3.1.3 Laterally Varying Structure. The regionalization of crustal structure described in Section 5.1 implies the existence of lateral variations in crustal structure at the boundaries of the regions. Although the nature of these boundaries has not been explored in most regions, there are several locations in the EUS in which they have. One of these locations is the New Madrid Rift area, in which the New Madrid region is surrounded by transitions in crustal structure to the south (Gulf Coast Plain); east (West Tennessee); north (Mid-continent); and west (Ozarks). Another is in Virginia, where

there is a very large thickening of the crust between central Virginia in the Atlantic Coastal Plain region and Giles County in the Southern Appalachian region.

In the following, we assess whether the variability in attenuation calculated from these variations in crustal structure within a region provides a realistic estimate of the variability in attenuation in data recorded within that region.

In a previous study, Saikia et al. (1992) analyzed the attenuation of vertical component S-waves from four small earthquakes in the New Madrid Rift area. Three of the four earthquakes occurred near the boundary of the New Madrid Rift, while one occurred within the Rift, and all recordings were made within the New Madrid Rift region, as shown in Figure 5-28. The three events were the 8 April 1981 Greenville, Illinois event at a depth of 5 km near the boundary with the Mid-continent region; the 7 August 1981 Dyersburg, Tennessee event at 23 km near the boundary with the West Tennessee region; and the 19 January 1983 Barney, Arkansas event at 4 km near the boundary with the Ozarks region. The event within the New Madrid region occurred near Equality, Illinois on 29 June 1984 at a depth of 7 km.

Although the paths of these recordings mostly lie within the New Madrid region, the thickness of the crust varies from about 40 to 47 km within the region. The attenuation of the recorded peak acceleration and velocity are compared with that of synthetic seismograms calculated using generalized ray theory for each event in Figures 5-29 and 5-30. Separate crustal structure models, shown in Figure 5-31, were used in calculating the attenuation for each earthquake.

The large variations in the shape of the calculated attenuation from event to event are due to variations in crustal structure and source depth. Similar variations exist in the empirical attenuation from event to event, which has an additional variability due to site effects (which are not modeled and are thus absent in the synthetics). To quantify the event-to-event variability in attenuation we have measured the standard deviation of the peak acceleration and velocity from a $1/R$ attenuation for each event in the distance range of 0 to 300 km for both the recorded and synthetic values. The results are summarized in Table 5-13. The standard deviations vary from 0.19 to 0.28 for the recorded peak velocities, and from 0.19 to 0.34 for the recorded peak accelerations. These variations are comparable to those of the simulations, which are from 0.11 to 0.34 for peak velocities and from 0.10 to 0.34 for peak accelerations. We conclude that in the

New Madrid rift region, the variability in ground motion attenuation due to variations in crustal structure may be reasonably estimated from synthetic attenuation relations derived from representative variations in crustal structure.

A much larger change in crustal structure occurs across the boundary between the Atlantic Coastal Plain and Southern Appalachian Regions in Virginia. An interpretation of the crustal structure of the area by Pratt et al. (1988) along a west-northwest profile is shown in Figure 5-32. A contour map of crustal thickness, derived using the time term method by James et al. (1968), is shown in Figure 5-33. The thickness of the Moho increases from about 30 km in the vicinity of Chesapeake Bay to 55 km in Giles County, over a distance of about 300 km. A profile across this crustal thickness gradient was chosen to coincide with the locations of small earthquakes recorded on the VPI Network shown in Figure 5-34. The crustal structure across this cross section was approximated using a horizontally layered upper crust and a planar Moho dipping at about 5 degrees to the west (Figure 5-35). As an alternative to a flat Conrad layer at a depth of 15 km, a Conrad layer that dips to the east at about 5 degrees is also shown.

A systematic difference is observed in the attenuation of ground motion amplitudes in either direction along this profile. For events in Central Virginia, the attenuation is rapid in the thickening direction toward Giles County, but for events in Giles County, the attenuation in the thinning direction toward Central Virginia is gradual. These trends persist after station terms have been removed, as described by Saikia et al. (1992). Using generalized ray synthetic seismograms, we have examined the influence of crustal models having flat and dipping Moho interfaces on the attenuation of ground motion in either direction along this profile. The results are summarized in Figures 5-36 and 5-37. Using flat layered models of the crust at the source location at either end of the profile does not produce a difference in attenuation. Introducing a dipping Moho flattens the attenuation toward Central Virginia at distances of 150 to 300 km, but not enough to match the flattening in the recorded data. The dipping Moho has relatively little effect on the attenuation toward Giles County. Introducing a Conrad layer that dips in the opposite direction to the Moho, as suggested by the interpreted profile shown in Figure 5-32 and shown as a dashed line in Figure 5-35, produces additional flattening of the attenuation toward Central Virginia, but still not enough to match the recorded data. The dipping Conrad also causes more gradual attenuation toward Giles County, which is not observed in the recorded data. The addition of crustal multiples has little effect on the attenuation.

In summary, we observe gradual attenuation toward Central Virginia and rapid attenuation toward Giles County. Using flat layered models of the crust at the source location at either end of the profile, we qualitatively reproduce the observed trend, but quantitatively the observed difference is greater than that simulated. Including a dipping Moho enhances the agreement with the data. We conclude that the variability in ground motion attenuation due to lateral variation in crustal structure in Virginia is larger than would be estimated from the variability in attenuation derived from flat layered crustal models that span the range of structure model variation. This is contrary to the conclusion that we reached for the New Madrid data, and may reflect the much larger degree of variation and complexity in crustal structure in Virginia than in New Madrid. The difference in crustal structure between Central Virginia and Giles County may be large enough to warrant using a separate ground motion attenuation region in the southern Appalachians in which Giles County is located, as described in Section 5.5.

5.3.2 *Inter-Regional Variation (For Fixed Q)*

A subset of the 16 regions shown in Figure 5-12 is used to examine the inter-regional velocity structure variation on ground motion attenuation. The subset consists of four regions that represent the extremes of the full 16 regions. These four are New Madrid, Gulf Coast, Grenville, and Western Tennessee. The ground motion attenuation for the 4 regions are computed using numerical simulations for a fixed Q model to isolate the effects of the velocity structure on the ground motion. The variability of the ground motion due to variation in focal depth for each velocity structure is shown in Figures 5-38a and b for 5 and 15 Hz. In these figures, the thin line shows the average standard error of ground motion using region specific mean attenuation for each region. The heavy line shows the standard error of the ground motion using a single region independent mean attenuation for all four regions. The difference between these two curves indicates the potential reduction in the standard error that can be realized by regionalizing the attenuation. The main difference is in the distance range 80 to 150 km due to variations in the Moho depth for the four regions.

5.4 Effects of Crustal Q and Kappa Structure

The effect of variation in Q and kappa on ground motion is examined using the generalized ray simulation procedure described in Section 3.3. The effect of variation in Q is analyzed for both intra-region variations and inter-region variations.

5.4.1 Effect of Variations in Q on Ground Motion

The effect of intra-region variation of Q on ground motion attenuation is estimated using three Q models developed for the New Madrid velocity structure (see Appendix 5A). The ground motions were computed for each Q model. The resulting variability of the ground motion is shown in Figure 5-39 for distances of 50, 100, and 200 km. The effect of uncertainty in Q becomes significant at distances greater than 100 km.

The effect of inter-region variation of Q on ground motion attenuation depends on the regions compared. For fifteen of the sixteen regions, the Q models are similar and the effect of inter-region variation of Q is small. This is not the case for the Gulf Coast region. Using the Mid-continent region as a representative crustal structure, the ground motions for the Mid-continent region for Q models 4 and 8 are shown in Figure 5-40a, b, c. The lower Q values for Q8 result in about a factor of 2 decrease in ground motion at frequencies of 5-15 Hz at distances greater than 100 km.

5.4.2 Effects of Variation in Kappa on Ground Motions

Although kappa is a site parameter, the kappa value for rock is effectively part of the path. Strong ground motion observed at hard rock sites in eastern North America appears to have different spectral properties than those typical of western North American rock motions. The differences in spectral content can perhaps be most easily seen in 5% damped spectral amplification (S_a/a) computed from recordings typical of WNA and ENA tectonic environments. Figure 5-41 shows average spectral shapes computed from recordings made on rock of a Nahanni earthquake (M 6.8) compared to an average spectral shape computed from M 6.5 western US rock motions (1971 San Fernando, M 6.6 and the 1979 Imperial Valley, M 6.4 earthquakes). The differences in spectral content are significant and indicate that, for the same level of peak acceleration, ENA spectral content is higher than that in western North America for frequencies greater than 10 Hz.

Other sources of data also indicate that ENA ground motion recorded at hard rock or very shallow soil sites is richer in high frequency energy relative to analogous western US recordings. These include aftershocks of the 1982 Miramichi, New Brunswick event (Cranswick et al., 1985), the 1982 Enola, Arkansas swarm (Harr et al., 1984), and the 1986 Northeastern Ohio event and its aftershocks (Wesson and Nicholson, 1986).

The trends shown in these data are consistent with the rock or shallow stiff soil recordings of strong ground motion. They indicate a significant and reasonably consistent difference in high frequency spectral content between representative rock motion in western and eastern North America crust.

In the stochastic model used here, (Section 3.2) the differences in spectral content between WNA and ENA at distances less than about 50 km is attributed to near surface crustal damping (Anderson and Hough, 1984; Silva and Darragh, 1993). The near surface damping parameter, κ , is used to model the effects of rate independent damping in the crustal rocks in the upper 1-2 km of the crust. Typical values for this parameter in western North America are about 0.03-0.04 sec (Silva and Darragh, 1993) and about 0.006 sec for hard rock ENA sites (Boore and Atkinson, 1987; Silva and Darragh, 1993).

To illustrate the differences in spectral content predicted by the stochastic model for western and eastern North American rock motions, the M 6.5 empirical spectral shapes shown in Figure 5-41 are modeled using Q and κ values appropriate for either ENA or WNA holding other source parameters constant. For WNA the stress drop is taken as 100 bars, the $Q(f)$ model is $150 f^{0.6}$, and κ is 0.040 sec (Silva and Darragh, 1993). To model the M 6.5 ENA spectral shape, the stress drop is also taken as 100 bars, but with the $Q(f)$ model of $670 f^{0.33}$ (Atkinson, 1993), and the κ value of 0.006 sec. Figure 5-42a shows the predicted spectral shape with ENA, Q , and κ and the average Nahanni spectral shape. Figure 5-42b shows the predicted spectral shape with WNA, Q , and κ and the average San Fernando and Imperial Valley spectral shape. As can be seen from these figures, the stochastic model captures the spectral shapes for both regions throughout most of the frequency range. The overprediction of the spectral shape computed from recordings of the Nahanni M 6.8 earthquake at periods shorter than about 0.05 sec may be an artifact of the recording instrument. The strong motion accelerographs (SMA-1) have corner frequencies near 0.04 sec. While instrument corrections were performed, they are notoriously difficult to correct with

confidence through an instrument corner unless very accurate phase and amplitude calibrations are available.

Figures 5-43 and 5-44 show response spectral shapes representative of ENA motions recorded at hard rock sites and stochastic model predictions for magnitude ranges of M 5.4-5.7 and M 4.0-4.8 respectively. The magnitude 5.4-5.7 range includes 4 recordings from 2 events and the magnitude 4.0-4.8 range includes 22 recordings from 3 events. The model predictions with a kappa value of 0.006 sec captures the overall spectral composition of the observed motions with reasonable accuracy.

The standard deviation is assumed to be 0.4 natural logarithm units based on the range of kappa values estimated by regression for 20 rock site recordings of the 1989 Loma Prieta Earthquake (Schneider et al. 1993). This variability in the kappa values from the WUS is assumed to be applicable to the kappa values from the EUS.

The variability of ground motion due to variability in kappa values is shown in Figure 5-45 for a site at a distance of 50 km. (Since kappa is a site term, the results are not sensitive to the distance.) For soft-rock sites, the variability of kappa has a significant effect on the high frequency ground motion variability; however, for hard-rock sites, the variability in kappa does not have a significant effect on the ground motion because the median kappa value is small.

5.5 Ground Motion Regionalization

The objective of this section is to determine how the regional variation of velocity structure, Q , and focal depth, discussed previously can best be implemented into a general attenuation relation for modeling ground motion attenuation in the EUS.

5.5.1 Approach

The philosophy used in determining the number of regions with distinct ground motion attenuation considered 4 factors: the ground motion differences should have both statistical and practical significance; the differences should be systematic; and the modeled differences should be robust.

For the ground motion differences to be of statistical significance, variations in the mean ground motion must be resolvable given the randomness of the focal depth and the

uncertainty of the Q model for each region. For the differences to be of practical significance, variations in the mean ground motion must be large enough to warrant modeling them given the size of the uncertainty of ground motion within a region. For the differences to be systematic, they should be consistent over a range of distances and periods.

For each of the 16 regions shown in Figure 5-12, numerically simulated ground motions were generated using the generalized ray method described in Section 3.3. These simulations were examined to determine the number of regions with distinct ground motion attenuation.

A non-parametric attenuation relation was estimated for the response spectral values for each frequency and region. Each frequency was fit independently and the variability in focal depth and intra-regional variation in Q were included. This gave a mean attenuation relation and the variance of the mean for the log spectral values. This variance represents the intra-regional variation. The non-parametric relations were computed at 10 km distance intervals and at the following frequencies: 1.0, 1.5, 2.5, 3.45, 5.0, 7.5, 10, 15, 25, and 34 Hz. These non-parametric relations were compared to each other to determine the number of distinct regions. The criteria for determining distinct regions is given below.

5.5.2 Description of Criteria

The initial criteria used to define significant differences in ground motion are as follows:

1. The differences in the medians must be significant at the 95% confidence level.
2. The difference in the medians must be greater than 20 percent.
3. Criteria 1 and 2 must be met for 3 consecutive distances and frequencies and the polarity of the difference in the medians must also be consistent over this distance and frequency range. (For this analysis, the distance criterion corresponds to a 30 km range.)

The robustness of the modeled differences is checked by varying these criteria and by using different focal depth distributions.

5.5.3 Results

Examples of the median spectral acceleration at 5 Hz for each of the 16 regions is shown in Figure 5-46 for the EAA depth distribution for combined margins and non-margins (denoted h_2). These ground motions include effects of both velocity structure and Q . This figure shows that some of the regions exhibit different attenuation from the average. For example, region 4 (Gulf Coast) has very different attenuation. The attenuation relations for all of the 16 regions are compared quantitatively below.

For the initial comparison, the attenuation for each region is compared to the attenuation for every other region. The results of this initial comparison of the attenuation for the 16 regions is summarized in a correlation matrix shown in Table 5-14. In this table, a 1 indicates that the attenuation for the pair of regions is not significantly different according the defined criteria (correlated), whereas a 0 indicates that attenuation is significantly different (uncorrelated).

Using the correlation matrix, the regions are combined into larger groups with similar (not distinct) attenuation. This grouping is done with the goal of finding a base group that contains as many regions as possible. The remaining regions are then grouped by minimizing the total number of groups needed to include all the regions. This initial grouping led to 3 attenuation groups listed below.

Attenuation Group	Regions
1	1, 2, 3, 5, 8, 9, 10, 12, 14, 15, 16 (6,11)
2	4
3	7, 13, (6,11)

Regions 6 and 11 are on the border between groups 1 and 3.

For each group, the median ground motion is computed. Regions 6 and 11 are excluded from the group average for both group 1 and group 3 since they are borderline regions. The attenuation for the individual regions is then compared to average attenuation for each group and a new correlation matrix is produced showing the correlation between the attenuation for each region and the average attenuation for each group. To test the robustness of the correlations, this process was repeated for alternative depth

distributions and different significance criteria. The alternative depth models are described in Appendix 5B.

The preferred depth distribution developed in Section 5.2.3 which distinguishes between margins and "other than margins" is depth distribution model h3 in table 5-15. These two depth distributions (margins and "other than margins") form a single depth distribution model. Regions 1,3, and 4 are considered margins and the remaining regions are "other than margins". The generic depth distribution developed in Section 5.2.3 which does not distinguish between margins and "other than margins" is depth distribution model h1 in Table 5-15. In considering multiple depth models, we do not compare the ground motions computed for alternative depth distribution models, but rather consider the differences in attenuation between regions for a given depth distribution model.

The results using the initial criteria are summarized in Table 5-15. This table shows that regions 4, 7, 13, and 16 are distinct from the base group (G1) for all of the depth distributions considered. Region 4 defines its own group (G2) which is distinct from all other regions. Group 3, defined by regions 7 and 13 is indistinct from a few regions in the base group for some depth distributions. Region 16 is distinct from all three groups for all depth distributions; however, as will be shown below this result is not robust.

The attenuation of spectral acceleration at 5 Hz for the three groups is plotted in Figure 5-47. The difference between G1 and G2 is quite strong with G2 yielding less attenuation at short distances and more attenuation at large distances. The difference between G3 and G1 is primarily at the distance range of 80 to 150 km and is likely related to the depth of the Moho for regions 7 and 13.

To test the robustness of the regionalization, the criteria for determining distinct regions was varied. For small changes in the criteria, region 16 became indistinct from G1. For these same variations in criteria, regions 7 and 13 remain distinct for most of the depth distributions and region 4 remains distinct from G1 for all depth distributions.

The distinction between the attenuation for G1 and G3 is not as strong as the distinction between attenuation for G1 and G2. The difference between G1 and G3 is restricted to a limited distance range (80-150 km) and the ground motions for G3 are lower than the ground motions for G1 over this distance range. The G1 model yields a conservative

estimate of the ground motion for G3. Therefore, for this project G3 is lumped with G1. As a result, two distinct ground motion attenuation regions are defined: the Gulf Coast (G2) and everything else (G1). Possible benefits of including a third attenuation region for G3 are discussed in Section 5.5.4.

5.5.3.1 Selection of Representative Velocity Models. A velocity model representative of group 1 attenuation was found by comparing the attenuation of each of the regions within group 1 with the average attenuation for group 1. The variance of the difference between the ground motion for each region and the ground motion for the group average was computed. Regions 3 and 12 gave the smallest variances for multiple depth distributions. An example of the attenuation for these two regions is shown in Figure 5-48 for peak acceleration. Region 12 was selected as the representative velocity model for group 1. Group 2 is simply defined by region 4.

The attenuation of ground motion for the two distinct regions is compared in Figures 5-49a,b,c. The Gulf Coast region (G2) has larger motions at short distances and lower motions at large distances compared with the Mid-continent region (G1). The Gulf Coast attenuation does not exhibit a flattening of the attenuation relation at 80 to 130 km that is seen for the Mid-continent region.

5.5.4 Possible Benefits of Finer Regionalization

Although group G3 was not used in this study, for sites in these regions there are possible benefits to using a finer regionalization that includes a distinct attenuation region, G3. In particular, the attenuation for G3 produces lower ground motion for the distance range of 80 to 150 km. If this distance range is of importance to a particular site then it would be beneficial to use a region specific ground motion model.

For G3, the ground motion difference at distances of 80 to 150 km is up to a factor of 1.5 smaller than for the base model. The effect on the hazard depends on the location of the seismic sources; therefore, the decision to develop a G3 attenuation relation would be a site-specific consideration.

5.5.5 Treatment of Regions With Low Near-Surface Velocities

The coastal margin of the Gulf Coast from the Mississippi Delta to East Texas has lower seismic velocities and a higher velocity gradient in the upper crust than those of the

general crustal model that was used to represent the Gulf Coast Region as a whole. Therefore, the Gulf Coast ground motion attenuation relation developed in Section 9 may not be applicable to the coastal margin of the Gulf Coast. A crustal model that represents the coastal margin was developed by replacing the 7 km thick top layer of the initial model with two layers. The revised model is listed together with the general model in Table 5-16. Sites in the coastal margin of the Gulf Coast should consider the effects of this coastal margin velocity structure model on the predicted ground motions.

The coastal margin may have some amplifications above the G2 model ground motions due to low velocities near the surface. One alternative approach is to use the G1 rock attenuation with the deep soil site amplifications from Section 6.

5.6 Q Models for the Two Attenuation Regions

The analysis of regional differences in ground motion attenuation in Section 5.5 used a regionalization of the Q that was self-consistent but not necessarily consistent with the semi-empirical simulation procedure. This was acceptable in that analysis because we were interested in the relative differences in the ground motion attenuation and not in the absolute values. However, this is not the case when the simulated motions are used to derive an attenuation relation (Section 9). Therefore, Q models that are consistent with the geometrical spreading term used with the stochastic simulation procedure are developed for the two attenuation groups (G1 and G2) determined in Section 5.5.

The background region, G1, is represented by the Mid-continent velocity structure. Q models for the EUS from various studies that used geometrical spreading terms similar to that computed for the Mid-continent velocity structure at large distances are shown in Figure 5-50. Based on these studies, median, high, and low Q models were developed. They are shown by the dashed lines in Figure 5-50. The selected Q models used in the simulations are listed in Table 5-17.

Region G2 is represented by the Gulf Coast velocity structure. Fewer Q models from other studies are available for this region. Two Q models from other studies using geometrical spreading terms similar to the stochastic model are also shown in Figure 5-51. For comparison, the Q model estimated for the Loma Prieta earthquake by inversion of the Fourier amplitude spectra (Section 3) is also shown. The Q for the Gulf Coast is closer to a WUS Q than it is to typical EUS Q. Based on these models, median, high, and

low Q models were developed. Median, high, and low Q models are shown by the dotted lines in Figure 5-51. The selected Q models used in the simulations are listed in Table 5-17.

5.7 References

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Table 5-1

List of Earthquakes Used in Ground Motion Attenuation Modeling

Location	Date	Lat	Long	Depth	Mag	# stations < 80 km <120		Crustal Model
West Quebec Ottawa	83 10 11	45.20	-75.750	15.0	4.1	3	3	Grenville Ontario
West Quebec*	87 11 11	45.776	-75.336	21.1	3.2	3	5	Grenville Ontario
West Quebec*	88 03 10	46.34	-75.67	12.	3.7	2	4	Grenville Ontario
Charlevoix	88 03 13	47.445	-70.376	6.8	3.1	7	7	Grenville Saguenay
West Quebec	88 08 09	45.007	-74.993	9.5	3.4	2	4	Grenville Ontario
Charlevoix Saguenay	88 11 25	48.117	-71.183	26.0	6.5	2	8	Grenville Saguenay
Charlevoix	89 11 22	47.445	-70.345	8.1	3.4	7	7	Grenville Saguenay
Charlevoix*	90 03 03	47.856	-69.977	20.0	3.6	7	7	Grenville Saguenay
Charlevoix*	90 04 21	47.553	-70.070	9.6	3.1	7	7	Grenville Saguenay
West Quebec Mont Laurier	90 10 19	46.474	-75.591	13.0	5.1	1	2	Grenville Ontario
Central Virginia	87 04 11	37.664	77.903	8.7	1.7	6	6	Atlantic Coastal Plain
Central Virginia	87 04 14	37.862	77.543	11.7	0.6	4	7	Atlantic Coastal Plain
Giles County	86 08 09	36.787	-80.726	10.5	2.2	5	7	Southern Appalachians
Giles County	87 11 21	36.755	-80.713	5.0		4	6	Southern Appalachians
East Tennessee	88 01 08	35.290	-84.203	15.0	2.3	4	7	Southern Appalachians
East Tennessee	88 02 11	35.468	-84.311	20.3	2.3	6	7	Southern Appalachians
New Madrid	91 05 04	35.555	-89.802	7.1	4.6	3	5	New Madrid- McCamy

Table 5-2

**Comparison of Recorded and Synthetic PSRV
Standard Errors of Log₁₀ Residuals for Brune Source Spectrum**

Event	1.0 Hz	2.2 Hz	4.8 Hz	9.0 Hz
83/10/11	0.224	0.140	0.289	0.183
87/11/11	0.102	0.153	0.178	0.162
88/03/10	0.066	0.060	0.107	0.147
88/03/13	0.278	0.271	0.262	0.214
88/08/09	0.171	0.339	0.379	0.317
88/11/25	0.367	0.298	0.246	0.204
89/11/22	0.338	0.316	0.451	0.403
90/03/03	0.309	0.280	0.261	0.291
90/04/21	0.157	0.121	0.198	0.194
90/10/19	0.126	0.311	0.267	0.222
87/04/11	0.185	0.208	0.261	0.347
87/04/14	0.254	0.256	0.263	0.322
87/11/21	0.282	0.213	0.399	0.451
88/01/08	0.234	0.168	0.196	0.278
88/02/11	0.127	0.173	0.103	0.165
91/05/04	0.064	0.125	0.125	0.261

Table 5-3

Crustal Velocity Structure Regions and Correlation with Tectonic Domains

Crustal Velocity Structure Regionalization		Tectonic Domains (Genetic Regionalization)	
No.	Name	No.	Name
1.	Offshore New England	218	Margin
2.	Northern Appalachians	221	Magdalen
3.	Atlantic Coastal Plain	218	Margin
4.	Gulf Coast Plain	216	Gulf Coast
		225	Ouachita
5.	Southern Appalachians	223	Piedmont
		224	Valley & Ridge
6.	Central Tennessee	223	Piedmont
		224	Valley & Ridge
		226	Grenville
7.	Western Tennessee	230	Granite-Rhyolite - East
8.	New Madrid Rift	228	Reelfoot Rift
		229	Cottage Grove - Rough Ck.
9.	Ozarks	230	Granite-Rhyolite - East
10.	Northern Grenville - Superior	236	Superior
		226	Grenville
		227	St Lawrence - Ottawa
		222	Acadia
11.	Lake Superior Basin	235	Keweenawan
12.	Mid-continent	230	Granite-Rhyolite - East
		233	Panokean
13.	Northern Great Plains	237	Trans Hudson
14.	Central Plains	255	Central Plains
15.	Southern Great Plains	231	Granite-Rhyolite - West
		232	Oklahoma Aulacogen
16.	Williston Basin	238	Wyoming

Table 5-4

Correlation of Q Regions to Velocity Structure Regions

Velocity Structure Region Q Region

1	1
2	1
3	1
4	8
5	4
6	4
7	4
8	6
9	2
10	4
11	2
12	2
13	2
14	1
15	2
16	1

Table 5-5

Q Values from Gupta et al. (1989)

Freq Band	Region 1		Region 2		Region 4		Region 6		Region 8	
	Q	σ	Q	σ	Q	σ	Q	σ	Q	σ
0.75 - 1.75	1046	165	1226	66	1961	217	1264	227	223	43
1.5 - 2.5	832	70	984	29	1281	62	948	86	315	56
2.0 - 4.0	1045	79	1229	32	1426	57	1020	73	503	109
3.0 - 5.0	1231	100	1487	42	1521	61	1304	103	605	166
4.0 - 6.0	1381	123	1684	54	1773	77	1385	120	605	166
5.0 - 7.0	1581	197	1999	80	2200	114	1671	205	660	253
6.0 - 8.0	1299	186	2416	119	2339	137	2706	604	2065	5091

Table 5-6

Q Models

Q Region	Model
1	600 f ^{0.50}
2	610 f ^{0.65}
4	850 f ^{0.50}
6	600 f ^{0.60}
8	180 f ^{0.80}

Table 5-7

**Depth Distributions of Earthquakes in Eastern North America (ENA)
and Eastern North America, Australia, and Africa (EAA)**

Region: Type:	ENA All	EAA All	ENA Rift	ENA Margin	ENA Other	EAA Rift	EAA Margin	EAA Other
DEPTH	No.	No.	No.	No.	No.	No.	No.	No.
1	0	1						1
2	0	0						
3	1	4	1			1		3
4	0	2				1		1
5	1	8			1			8
6	0	6					1	5
7	2	7		1	1	1	1	5
8	2	3			2	0		3
9	2	2	1		1	1		1
10	4	21			4	2	5	14
11	2	4		2			3	1
12	1	2			1			2
13	0	2					1	1
14	1	4			1	1	1	2
15	2	7	1		1	1	2	4
16	1	2		1			1	1
17	0	1						1
18	4	5		3	1	1	3	1
19	2	3	1	1		1	1	1
20	0	2					1	1
21	0	0						
22	0	0						
23	0	1						1
24	0	0						
25	1	2			1		1	1
26	0	2					2	
27		1		0		1		
28	1	3			1	1	1	1
29	1	1		1			1	
30	1	2		1			1	1
31	1	0		1				
32	0	0						
33		0						
34		1						1
35		1						1
36		0						
37		1					1	
38		0						
39		0						
40		0						
41		1						1
42		0						
43		1						1

Table 5-8

Weights for Depth Distribution Model (EAA)

Depth	Margins	Non-Margins
5	0.077	0.352
10	0.308	0.338
15	0.192	0.155
20	0.192	0.070
25	0.115	0.042
30	0.115	0.042

Table 5-9

Weights for Generic Depth Distribution Model (EAA)

Depth	Weight
5	0.216
10	0.320
15	0.171
20	0.070
25	0.077
30	0.019

Table 5-10

Intra-Region Crustal Models for the Northern Grenville - Superior Region

Grenville - West Quebec

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
5.0	2.34	2.53	1.0	0
6.3	3.64	2.74	3.0	1.0
6.4	3.70	2.76	2.0	4.0
6.5	3.76	2.78	2.0	6.0
6.6	3.82	2.79	12.0	8.0
6.7	3.87	2.81	10.0	20.0
6.8	3.93	2.83	8.0	30.0
8.1	4.68	3.35	--	38.0

Grenville - Gradient

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
5.0	2.34	2.53	1.0	0
6.3	3.64	2.74	3.0	1.0
6.4	3.70	2.76	2.0	4.0
6.5	3.76	2.78	2.0	6.0
6.6	3.82	2.79	12.0	8.0
6.7	3.87	2.81	10.0	20.0
6.8	3.93	2.83	3.0	30.0
7.0	4.05	2.86	3.0	33.0
7.2	4.16	2.89	2.0	36.0
7.4	4.28	2.92	2.0	38.0
7.6	4.39	2.95	2.0	40.0
7.8	4.51	2.99	1.0	42.0
8.0	4.62	3.35	--	43.0

Grenville - Charlevoix

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.5	2.6	2.30	1.44	0
5.5	3.4	2.50	6.0	1.44
6.1	3.5	2.67	12.0	7.44
6.6	3.7	2.85	14.0	19.44
7.0	4.0	3.02	10.0	33.44
8.2	4.7	3.35	--	43.44

Grenville - Berry

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
5.0	2.34	2.53	2	0
6.3	3.65	2.74	17	2
6.8	3.93	2.83	22	19
8.2	4.74	3.35	--	41

Table 5-11

Intra-Region Crustal Models for the New Madrid Rift Region

New Madrid - McCamy

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.7	2.72	2.48	4	0
6.2	3.58	2.73	5	4
6.5	3.76	2.78	21	9
7.4	4.28	2.92	15	30
8.1	4.68	3.35	--	45

New Madrid - Flank

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	3.0	0.0
6.2	3.58	2.73	11.0	4.0
6.6	3.82	2.79	18.0	14.0
7.3	4.22	2.91	9.0	32.0
8.0	4.62	3.35	--	41.0

New Madrid - Ginzburg

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	5	0
6.2	3.58	2.73	9	5
6.6	3.82	2.79	12	14
7.3	4.22	2.91	20	26
8.0	4.62	3.35	--	46

New Madrid - Saikia

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
3.60	2.10	2.20	0.50	0
5.60	3.28	2.50	1.00	0.5
6.15	3.60	2.70	8.00	1.5
6.40	3.70	2.90	10.00	9.5
7.45	4.20	3.00	20.00	19.5
8.18	4.78	3.34	--	39.5

Table 5-12

Crustal Models Representing Variability in Poisson's Ratio

Mid-continent - Base Case

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.83	2.52	1	0
6.1	3.52	2.71	11	1
6.5	3.75	2.78	28	12
8.0	4.62	3.35	--	40

Mid-continent - Slow S Wave Velocity

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.69	2.52	1	0
6.1	3.35	2.71	11	1
6.5	3.57	2.78	28	12
8.0	4.40	3.35	--	40

Mid-continent - Fast S Wave Velocity

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.9	2.95	2.52	1	0
6.1	3.69	2.71	11	1
6.5	3.91	2.78	28	12
8.0	4.82	3.35	--	40

Table 5-13

Variability* in Ground Motion Attenuation within the New Madrid Region

Event Date and Location	Peak Velocity Recorded	Peak Velocity Synthetic	Peak Acceleration Recorded	Peak Acceleration Synthetic
81.04.08 Greenville, IL	0.25	0.24	0.26	0.29
81.08.07 Dyersburg, TN	0.28	0.11	0.34	0.10
83.01.19 Barney, AK	0.19	0.21	0.19	0.25
84.06.29 Equality, IL	0.26	0.34	0.24	0.34

*expressed as a natural logarithm of standard error of values about a 1/R line

Table 5-14

Correlation Matrix for Ground Motion Attenuation Between Regions

Region	Regions															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	1	1	1	0	0	1	0	0	1	1	0	1	0	0	0	0
2	1	1	1	0	1	0	0	0	1	0	0	1	0	1	0	1
3	1	1	1	0	1	0	0	0	1	0	0	1	0	0	1	1
4	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
5	0	1	1	0	1	0	0	0	0	0	0	1	0	0	1	1
6	1	0	0	0	0	1	0	0	0	0	0	1	1	0	0	0
7	0	0	0	0	0	0	1	0	0	0	0	0	1	0	0	0
8	0	0	0	0	0	0	0	1	0	0	1	1	0	1	0	1
9	1	1	1	0	0	0	0	0	1	1	0	1	0	1	1	0
10	1	0	0	0	0	0	0	0	1	1	0	1	0	1	0	0
11	0	0	0	0	0	0	0	1	0	0	1	1	1	0	0	0
12	1	1	1	0	1	1	0	1	1	1	1	1	0	1	1	0
13	0	0	0	0	0	1	1	0	0	0	1	0	1	0	0	0
14	0	1	0	0	0	0	0	1	1	1	0	1	0	1	0	0
15	0	0	1	0	1	0	0	0	1	0	0	1	0	0	1	0
16	0	1	1	0	1	0	0	1	0	0	0	0	0	0	0	1

A 1 indicates the attenuation for the pair of regions is similar. A 0 indicates that the attenuation for the pair of regions is different.

Table 5-15
Correlation Matrices for Ground Motion Attenuation
Between Individual Regions and Group Averages

	h1			h2			h3		
Region	G1	G2	G3	G1	G2	G3	G1	G2	G3
1	0	0	0	0	0	0	1	0	0
2	1	0	0	1	0	0	1	0	0
3	1	0	0	1	0	0	0	0	0
4	0	1	0	0	1	0	0	1	0
5	1	0	0	1	0	0	1	0	0
6	0	0	0	1	0	1	0	0	0
7	0	0	1	0	0	1	0	0	1
8	1	0	0	0	0	1	1	0	0
9	1	0	0	1	0	0	1	0	0
10	0	0	0	0	0	0	0	0	0
11	1	0	1	0	0	1	1	0	0
12	1	0	0	1	0	0	1	0	0
13	0	0	1	0	0	1	0	0	1
14	1	0	0	0	0	0	0	0	0
15	1	0	0	1	0	0	1	0	0
16	0	0	0	0	0	0	0	0	0

Focal Depth Distributions (see Appendix 5B):

- h1 EAA (Generic)
- h2 ENA (Generic)
- h3 EAA (margins and other than margins) Margin Regions: 1,3,4
- h4 Atkinson (rifted and non-rifted) Rifted Regions: 8,11,2,10,12,14,15
- h5 Atkinson (rifted and non-rifted) Rifted Regions: 8, 11
- h6 Wheeler

Distinct Criteria:

1. Significant difference in mean at 95% confidence level.
2. Difference in median ground motion $\geq 20\%$.
3. Criteria 1 and 2 hold for 3 consecutive distances and frequencies with consistent polarity of difference.

Table 5-15 (continued)

Region	h4			h5			h6		
	G1	G2	G3	G1	G2	G3	G1	G2	G3
1	0	0	0	0	0	0	0	0	0
2	1	0	0	0	0	0	1	0	0
3	0	0	0	0	0	0	1	0	0
4	0	1	0	0	1	0	0	1	0
5	1	0	0	1	0	0	1	0	0
6	0	0	0	0	0	0	0	0	0
7	0	0	1	0	0	1	0	0	1
8	0	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	1	0	0
10	1	0	0	0	0	0	0	0	0
11	0	0	0	0	0	0	0	0	1
12	1	0	0	1	0	0	1	0	0
13	0	0	1	0	0	1	0	0	1
14	1	0	0	0	0	0	0	0	0
15	1	0	0	0	0	0	1	0	0
16	0	0	0	0	0	0	0	0	0

Focal Depth Distributions (see Appendix 5B):

- h1 EAA (Generic)
- h2 ENA (Generic)
- h3 EAA (margins and other than margins) Margin Regions: 1,3,4
- h4 Atkinson (rifted and non-rifted) Rifted Regions: 8,11,2,10,12,14,15
- h5 Atkinson (rifted and non-rifted) Rifted Regions: 8, 11
- h6 Wheeler

Distinct Criteria:

1. Significant difference in mean at 95% confidence level.
2. Difference in median ground motion $\geq 20\%$.
3. Criteria 1 and 2 hold for 3 consecutive distances and frequencies with consistent polarity of difference.

Table 5-16
Gulf Coastal Plain - General Model

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
4.0	2.31	2.37	7	0
5.3	3.05	2.58	8	7
6.5	3.76	2.7	15	15
8.2	4.74	3.40	--	30

Gulf Coastal Plain - Coastal Margin Model

Vp (km/sec)	Vs (km/sec)	Density (gm/cc)	Thickness (km)	Depth to Top
2.2	0.82	2.00	3	0
3.4	1.82	2.27	4	3
5.3	3.05	2.58	8	7
6.5	3.76	2.78	15	15
8.2	4.74	3.40	--	30

Table 5-17
Q Models Used with the Stochastic Simulation Procedure

	Mid-continent		Gulf Coast	
	Q ₀	η	Q ₀	η
high	1000	0.3	500	0.3
median	670	0.33	300	0.3
low	400	0.4	200	0.3

Figure 5-1. Schematic diagram of the wave propagation model, with direct waves (S) and waves ScS and SmS reflected from the Conrad and Moho layers. At the critical angle (i_{crit}), the incident wave is totally reflected back to the surface at distances starting at the critical distance R_{crit} .

Figure 5-2 (A). Top Frame: Empirical attenuation in southeastern Canada for Fourier amplitude at 2 Hz from Atkinson and Mereu (1992). Station corrections and source corrections have been applied. The solid curve is the regression fit to these data. The dashed and dotted lines are based on the stochastic model described in Section 3.2.

Figure 5-2 (B). Mean empirical attenuation in southeastern Canada for Fourier amplitudes.
Source: Atkinson and Mereu, 1992.

Figure 5-3. Map showing the locations of the ten earthquakes in the Northern Grenville-Superior region and the recordings stations used in the analysis.

Figure 5-4. Left: Crustal structure model for the Ottawa and Mont Laurier earthquakes, modified from Mereu et al. (1986). Right: Crustal structure model for the Saguenay and Charlevoix earthquakes, from Somerville et al. (1990).

Figure 5-5. Comparison of recorded (left) and synthetic (right) vertical velocity seismograms for the 03/11/88 West Quebec event, shown with predicted travel times for direct and reflected waves. To display S wave arrivals, the time scale is the travel time reduced by 4 km/sec.

Figure 5-6. Comparison of recorded (left) and synthetic (right) vertical velocity seismograms for the 11/11/87 West Quebec event, shown with predicted travel times for direct and reflected waves. To display S wave arrivals, the time scale is the travel time reduced by 4 km/sec.

Figure 5-7. Comparison of recorded (left) and synthetic (right) vertical velocity seismograms for the 4/21/90 Charlevoix event, shown with predicted travel times for direct and reflected waves. To display S wave arrivals, the time scale is the travel time reduced by 4 km/sec.

Figure 5.-8. Comparison of recorded (left) and synthetic (right) vertical velocity seismograms for the 11/25/88 Saguenay event, shown with predicted travel times for direct and reflected waves. To display S wave arrivals, the time scale is the travel time reduced by 4 km/sec.

Figure 5-9. Comparison of recorded and synthetic peak acceleration as a function of distance for ten Grenville region earthquakes. Recorded PSRV at 5% damping at 2 Hz is shown on the left, and synthetic values are shown on the right. The top plots are for the three deep events, and the bottom plots are for the seven shallow events. Least-squares fits to each event, normalized at 40 km, are shown by the straight lines. The $1/R$ attenuation is shown by the dashed line.

Figure 5-10. Residuals between recorded and synthetic response spectral velocity at 5% damping at 0.98, 2.2, 4.8, and 9.0 Hz, and peak acceleration and velocity, as a function of distance for ten Grenville region earthquakes. The open boxes are for shallow events (less than 20 km deep), while the crosses are for deep events.

Figure 5-11. Map showing the locations of the ten earthquakes in eastern North America used in the analysis.

Figure 5-12. Residuals between recorded and synthetic response spectral velocity at 5% damping at 0.98, 2.2, 4.8, and 9.0 Hz, and peak acceleration and velocity, as a function of distance for ten eastern North American earthquakes. The open boxes are for shallow events (less than 20 km deep), while the crosses are for deep events.

Figure 5-13. Profiles of crustal structure across the eastern United States at latitudes 49 and 37 degrees (Braille et al., 1989).

Figure 5-14. Crustal structure regionalizations for the EUS (Woodward-Clyde, 1991).

Regionalization Summary

Figure 5-15. Summary of velocity models for the 16 regions shown in Figure 5-12.

Figure 5-16. Crustal structure regionalization for eastern North America based on genetic origin.

Figure 5-17. Regionalization of Q models from the Gupta et al. (1989) study.

Figure 5-18 (A). Comparison of Q data from the Gupta et al. (1989) study with the parametric, model for region Q1. The 1 Hz values was not used in the regression because it did not fit the trend of the other frequencies and is at the lower bound of the frequency band of interest.

Figure 5-18 (B). Comparison of Q data from the Gupta et al. (1989) study with the parametric, model for region Q2. The 1 Hz value was not used in the regression because it did not fit the trend of the other frequencies and is at the lower bound of the frequency band of interest.

Figure 5-18 (C). Comparison of Q data from the Gupta et al. (1989) study with the parametric, model for region Q4. The 1 Hz values was not used in the regression because it did not fit the trend of the other frequencies and is at the lower bound of the frequency band of interest.

Figure 5-18 (D). Comparison of Q data from the Gupta et al. (1989) study with the parametric, model for region Q4. The 1 Hz values was not used in the regression because it did not fit the trend of the other frequencies and is at the lower bound of the frequency band of interest.

Figure 5-18 (E). Comparison of Q data from the Gupta et al. (1989) study with the parametric, model for region Q8. The 1 Hz values was not used in the regression because it did not fit the trend of the other frequencies and is at the lower bound of the frequency band of interest.

Figure 5-19. Comparison of the five Q models used to represent the Q for the 16 velocity regions.

Figure 5-20. Focal depth distributions for eastern North America, Australia and Africa from the EPRI Stable Continental Regions Catalog.

Figure 5-21. Separation of focal depths in the combined eastern North America, Africa, Australia data set according to tectonic type: rift, margin, and other. The distribution for margins has a deeper median depth than for rifts or other regions.

Figure 5-22. Depth distribution model for margins and non-margins based on the combined eastern North America, Africa, and Australia data set. This depth distribution model is considered in the regression analysis in Section 9.

Figure 5-23. Four velocity models used for the Grenville region to represent variability of the velocity model within a region.

Figure 5-24. Four velocity models used for the New Madrid region to represent variability of the velocity model within a region.

Figure 5-25 (A). Variability of spectral acceleration at 1 Hz due to crustal velocity uncertainty within the New Madrid region. The variability is computed separately for each focal depth.

Figure 5-25 (B). Variability of spectral acceleration at 5 Hz due to crustal velocity uncertainty within the New Madrid region. The variability is computed separately for each focal depth.

Figure 5-25 (C). Variability of spectral acceleration at 15 Hz due to crustal velocity uncertainty within the New Madrid region. The variability is computed separately for each focal depth.

Figure 5-26 (A). Variability of spectral acceleration at 1 Hz due to crustal velocity uncertainty within the Grenville region. The variability is computed separately for each focal depth.

Figure 5-26 (B). Variability of spectral acceleration at 5 Hz due to crustal velocity uncertainty within the Grenville region. The variability is computed separately for each focal depth.

Figure 5-26 (C). Variability of spectral acceleration at 15 Hz due to crustal velocity uncertainty within the Grenville region. The variability is computed separately for each focal depth.

Figure 5-27 (A). Variability of spectral acceleration at 1 Hz due to crustal velocity uncertainty within the Grenville and New Madrid region. The variability includes variability in focal depth.

Figure 5-27 (B). Variability of spectral acceleration at 5 Hz due to crustal velocity uncertainty within the Grenville and New Madrid region. The variability includes variability in focal depth.

Figure 5-27 (C). Variability of spectral acceleration at 15 Hz due to crustal velocity uncertainty within the Grenville and New Madrid region. The variability includes variability in focal depth.

Figure 5-28. Location of the four events in the New Madrid region. The contours indicate the depth of the Moho. For these events, the Moho depth varies from 40 to 47 km.

Figure 5-29. Observed and synthetic attenuation of peak acceleration for the four events in the New Madrid region. Straight lines are fits to the observed and synthetic values using a $1/R$ relation.

○ Data
● Synthetics
— Data regression
--- Synth regression

Figure 5-30. Observed and synthetic attenuation of peak velocity for the four events in the New Madrid region. Straight lines are fits to the observed and synthetic values using a $1/R$ relation.

Figure 5-31. Velocity models for the four events in the Grenville region.

Figure 5-32. Crustal structure cross section across the boundary between the Atlantic Coastal Plain and Southern Appalachian Regions in Virginia.

Figure 5-34. Earthquake locations and VPI network.

Figure 5-35. Two-dimensional crustal model from Giles County to Central Virginia. An alternative model with a dipping Conrad in addition to the dipping Moho is shown by the dashed line.

Figure 5-36. Attenuation of peak velocity from Giles County toward Central Virginia. The attenuation predicted by the synthetic motions is shown by the curves.

Figure 5-37. Attenuation of peak velocity from Central Virginia toward Giles County. The data show more rapid attenuation than from Giles County toward Central Virginia. The attenuation predicted by the synthetic motions is shown by the curves.

Figure 5-38 (A). Effect of inter-region crustal velocity variations on the variability of spectral acceleration at 1 Hz.

Figure 5-38 (B). Effect of inter-region crustal velocity variations on the variability of spectral acceleration 5 Hz.

Figure 5-38 (C). Effect of inter-region crustal velocity variations on the variability of spectral acceleration at 15 Hz.

Figure 5-39. Effect of variability of Q within a region on the variability of spectral acceleration.

Figure 5-40 (A). Effect of variability of Q between regions on the rate of attenuation of spectral acceleration at 1 Hz.

Figure 5-40 (B). Effect of variability of Q between regions on the rate of attenuation of spectral acceleration at 5 Hz.

Figure 5-40 (C). Effect of variability of Q between regions on the rate of attenuation of spectral acceleration at 15 Hz.

Figure 5-41. Comparison of an average spectral shape for a hard rock site (Nahanni) with an average spectral shape for WUS rock sites. This illustrates the difference between spectral shapes for typical hard rock sites in the EUS and rock sites in the WUS.

Figure 5-42 (A). Comparison of average spectral shapes for hard rock sites from the Nahanni event (Figure 5-41) with the predicted ENA shape based on the stochastic ground motion model. The stochastic model captures the spectral shapes for hard rock sites.

Figure 5-42 (B). Comparison of average spectral shapes for rock sites from WUS events (Figure 5-41) with the predicted WUS shape based on the stochastic ground motion model.

Figure 5-43. Comparison of recorded and predicted spectral shapes for ENA hard rock sites for $M 5.4 - 5.7$.

Figure 5-44. Comparison of recorded and predicted spectral shapes for ENA hard rock sites for M 4.0 - 4.8.

Figure 5-45. Effect of variability of kappa on the variability of spectral acceleration.

Figure 5-46. Comparison of median ground motion attenuation of spectral acceleration at 5 Hz for the 16 regions.

Figure 5-47. Comparison of median ground motion attenuation of spectral acceleration at 5 Hz for the three attenuation groups.

Figure 5-48. Comparison of G1 attenuation of spectral acceleration at 5 Hz with the attenuation for regions 3 and 12. Over all frequencies, the velocity structure for region 12 best approximates the attenuation for G1.

Figure 5-49 (A). Comparison of G1 and G2 attenuation of spectral acceleration at 1 Hz.

Figure 5-49 (B). Comparison of G1 and G2 attenuation of spectral acceleration at 5 Hz.

Figure 5-49 (C). Comparison of G1 and G2 attenuation of spectral acceleration at 15 Hz.

Figure 5-50. Q models for the Mid-continent region. The dashed lines are the median, high, and low Q models used to generate the ground motions analyzed in Section 9.

Q Models for the Gulf Coast Region

Figure 5-51. Q models for the Gulf Coast region. The dashed lines are the median, high, and low Q models used to generate the ground motions analyzed in Section 9.

SECTION 6
QUANTIFICATION OF SITE EFFECTS

QUANTIFICATION OF SITE EFFECTS

6.1 Introduction

The objective of this section is to develop and present soil amplification factors for 5% damped response spectra, peak acceleration, and peak particle velocity which may be applied to the ground motion model of Section 9 to accommodate the effects of soils on a generic basis. A set of five generic soil categories ranging in depth from 20 ft to 500 ft are presented as well as depth dependent modulus reduction and damping curves. One-dimensional equivalent-linear site response analyses are used to develop median and 1 sigma estimates of the soil response. The amplification factors are computed for control motions with Eastern North America characteristics and vary in level from 0.05g to 1.25g. In addition, preliminary results are presented for the vertical-to-horizontal (V/H) ratio at both rock and soil sites. Based on these results, guidelines are presented for computing vertical motions for the Midcontinent and Gulf Coast crustal structures as well as the five soil site categories.

A set of generic modulus reduction and damping curves reflecting the effects of confining pressure are used in the site response analyses. The curves are developed in Appendix 7.A based on laboratory testing results presented in Section 8.

The major issues in predicting the effects of site response to strong ground motion have been evaluated in Appendix 6.B. These issues include the adequacy of the vertically propagating shear-wave model, the in situ strain dependencies of dynamic material properties, and how the effects of material nonlinearities are treated computationally (i.e., nonlinear analyses versus equivalent-linear analyses). To evaluate these factors, nonlinear analyses using computer codes DESRA-2C, SUMDES, and TESS were compared to equivalent-linear analyses and to recorded strong ground motions at three reference sites; Gilroy #2, Treasure

Island, and the Lotung, Taiwan LSST site. The analyses used geotechnical site models based on laboratory testing of undisturbed samples and geophysical surveys. In addition, the analysis procedures were compared to each other using synthetic control motions ranging from 0.50 to 1.25g with the 120 ft Category 3 shear-wave velocity profile (Section 6.3.3).

The results of the analyses comparison and validation with recorded motions indicate that conventional one-dimensional site response analyses incorporating nonlinear soil behavior based upon careful laboratory testing and with reasonably accurate soil profiles can accurately predict the effects of soils. Additionally, for the profiles and motions studied, little difference was seen between fully nonlinear and equivalent-linear analyses. Based upon these results, the equivalent-linear approach is used in the following analyses.

6.2 Overview

Observations of the effects of the ground on shaking during earthquakes have a long history. Del Barrio, in the 1855 Proceedings of the University of Chile states^{*} "...a movement.... must be modified while passing through media of different constitutions. Therefore, the earthquake effects will arrive to the surface with higher or lesser violence according to the state of aggregation of the terrain which conducted the movement. This seems to be, in fact, what we have observed in the Colchagua Province (of Chile) as well as in many other cases" (Del Barrio, 1855). In 1862, Mallet (1862) noted the effect of geology upon earthquake damage. Milne (1908) observed that in soft "damp" ground it was easy to produce vibrations of large amplitudes and long duration, while in rock it was difficult to produce vibrations of sufficient amplitude to be recorded.

Wood (1908) and Reid (1910), using apparent intensity of shaking and distribution of damage in the San Francisco Bay area during the 1906 earthquake, gave evidence that the severity of shaking can be substantially affected by the local geology and soil conditions. Gutenberg (1927, 1957) developed amplification factors representing different site geology by examining

^{*}Translated from the old Spanish by Professor Ricardo Dobry.

recordings of microseisms and earthquakes from instruments located on various types of ground.

Figure 6-1 shows average spectral shapes (response spectral acceleration divided by peak acceleration) computed from recordings made on rock and soil sites at close distances to earthquakes in the magnitude range of about M_0 to 7. The differences in spectral shapes are significant and depend strongly upon the general site classifications. These variations in spectral content represent average site dependent ground motion characteristics and result from vertical variations in soil material properties (Hayashi et al., 1971; Mohraz, 1976; Seed et al., 1976). Due primarily to the limited number of records from earthquakes of different magnitudes, spectral content in terms of response spectral shapes was for some time interpreted not to depend upon magnitude nor distance, but primarily on the stiffness and depth of the local soil profile. However, with an increase in the strong motion database, it has become apparent that spectral shapes depend strongly upon magnitude as well as site conditions (Joyner and Boore, 1982; Idriss, 1985; Silva and Green, 1989), and distance (Silva and Green, 1989), and that site effects extend to rock sites as well (Boatwright and Astrue, 1983; Campbell 1981, 1985, 1988; Cranswick et al., 1985; Silva and Darragh, 1993).

Examples of differences in spectral content largely attributable to one-dimensional site effects at rock sites can be seen in comparisons of response spectral shapes computed from motions recorded in both active and stable tectonic regions (Silva and Darragh, 1993). Figure 6-2 shows average spectral shapes (S_a/a_{max}) computed from recordings made on rock at close distances to large and small earthquakes (Table 6-1). For both magnitudes (moment magnitude M 6.4 and 4.0), the motions recorded in Eastern North America (ENA), a stable tectonic region, show a dramatic shift in the maximum spectral shape toward higher frequencies compared to the Western North American (WNA) motions. These differences in spectral content are significant and are interpreted as primarily resulting from differences in the shear-wave velocity and damping in the rocks directly beneath the site (Boore and Atkinson, 1987; Toro and McGuire, 1987; Silva and Green, 1989; Silva and Darragh, 1993). Also evident in Figure 6-2 is the strong magnitude dependency of the response spectral shapes. The smaller earthquakes show a much narrower bandwidth. This is a consequence

of lower corner frequencies for smaller magnitude earthquakes (Boore, 1983; Silva and Green, 1989; Silva and Darragh, 1993).

The difference in spectral content due to soil site effects, as shown in Figure 6-1, and due to rock site effects, as shown in Figure 6-2, are dramatic and illustrate the degree to which one-dimensional site conditions (vertical variations in dynamic material properties) control strong ground motions.

Since the 1930's, a number of Japanese seismologists have made contributions to the theory of site response (e.g., Sezawa and Kanai, 1932; Kanai, 1950; Kanai et al., 1959; Tanaka et al., 1973). These contributions generally included observations of site amplification effects from small earthquakes and theoretical developments for solutions of the wave equation at sites with up to three layers. Input motion was assumed to be vertically propagating shear waves. A rate-dependent material damping was included through viscosity terms. These investigators generally found reasonable agreement between theory and observation in amplitude and predominant period of site resonances. Observations included simultaneous recordings at depth in mines and at the surface. In addition, microtremor observations were made at different locations and at different times. One of the main observations by these workers was that ground motion on soil deposits is generally higher than that occurring on adjacent rock outcrops. Their theoretical developments indicate that this observed phenomenon was likely caused by the lower wave velocities in soil deposits and resonances due to reverberations in the soil column. Some of these studies were incorporated into early engineering practice in Japan.

6.2.1 Recent Observations and Development

Haskell (1960) extended the early work of Thompson (1950), and introduced the elegant matrix operator formalism which allowed the treatment of compressional and vertically polarized shear (SV), as well as horizontally polarized shear (SH), wavefields with arbitrary angles of incidence and more general material damping. This frequency domain wave-propagation approach (Thompson-Haskell method) also increased computational efficiency, allowing many layers with constant material properties to be incorporated into the analyses.

This development coincided with the emergence of digital computers, and the combination permitted more accurate modeling of soil variability. This approach was subsequently generalized by Gilbert (Gilbert and Backus, 1966) to allow continuous changes in material properties with depth (Yamamoto, 1983). These procedures marked the early beginnings of computational seismology. At the time, however, they were not widely employed in the engineering community.

Paralleling these wave propagation developments, site response analyses were being performed for engineering purposes using a lumped-mass approach (Idriss and Seed, 1968). This formulation provided a useful representation for vertically propagating shear or compression waves. Early lumped-mass approaches suffered from the constraint of a rigid base (Moriwaki and Doyle, 1978). The rigid base trapped wave energy within the soil column, and hence reflected waves were not permitted to radiate into the half-space. This artificial limitation, in some cases, resulted in unrealistic site response analyses. The later development of the Lysmer-Kuhlemeyer dashpot (Lysmer and Kuhlemeyer, 1975) for finite element applications largely resolved this difficulty by allowing the rigid half-space to act like a linear elastic half-space.

In the early sixties, Duke and his co-workers at UCLA (Duke and Leeds, 1962) extended Kanai's formulation (Kanai, 1950) to an arbitrary number of layers which included viscosity terms. By the late sixties and the early seventies, seismologists were routinely utilizing the Thompson-Haskell formulation in crustal response analyses (e.g., Phinney, 1964; Randall, 1967; Knopoff, 1964; Kurita, 1973; Leblanc, 1967; Ishii and Ellis, 1970) and to infer crustal compression and shear-wave velocities. Borchardt (1971) and Silva (1976) extended the matrix formulation to enable the exact treatment of damping so that soils with high shear-wave damping could be modeled (Joyner, et al., 1976; Johnson and Silva, 1981).

These developments allowed more detailed and varied analyses of linear site response observations and provided a focus for further observations. For example, Borchardt and Gibbs (1976), Seed et al. (1969), Wiggins (1964), Idriss and Seed (1968), Berrill (1977), and Duke and Mal (1978) have shown that during small and large earthquakes the surface soil

motion can differ in significant and predictable ways from that on adjacent rock outcrops. In addition, other investigations have utilized explosion data either independently or in conjunction with earthquake data to examine site response characteristics (Murphy et al., 1971; Rogers et al., 1983; and Hays, et al., 1979). Recent work using horizontal as well as vertical arrays of instruments have demonstrated the general consistency of the site response for seismic events of different sizes, distances, and azimuths (Tucker et al., 1984; Benites, et al., 1985; Silva et al., 1988, Silva et al., 1991; Su et al., 1992; Silva and Stark, 1992).

Results of these and other studies have generally concurred that the motion at the surface of soft sites is greater than that at stiff sites for the same level of relatively low excitation. Additionally, these studies have demonstrated, in a general sense, the adequacy of assuming plane-wave propagation in modeling linear one-dimensional site response for engineering purposes. (It should be noted that one-dimensional modeling may not be appropriate where significant lateral heterogeneity exists; e.g., King and Tucker, 1984; Barel and Gariel, 1986).

The physical mechanism responsible for the observed soil amplification is an increase in wave amplitude as propagation velocity decreases toward the surface. If material damping is neglected, the decrease in shear-wave velocity toward the surface results in an increase in motions. This arises because flow of energy per unit time and per unit area (energy flux) $\rho V_s \dot{u}^2$ (ρ = density, V_s = propagation velocity, $\dot{u}$ = particle velocity; Aki and Richards, 1980) is conserved. Therefore if V_s decreases, $\dot{u}$ must increase in an elastic system. In any soil or rock column, however, some material damping is always present and the net amplification (or deamplification) involves an interplay between counteracting effects. This may result in some cases in an amplification of peak particle velocity and a deamplification of peak acceleration due to the different frequency content of the two measures of ground motion.

The other phenomenon which is observed, and is predicted by simple shear-beam theory, is the presence of resonances. These are due simply to the constructive interference of upgoing and downgoing waves in the soil column. Since there is a stiffness contrast at the soil/rock boundary, a fraction of the seismic energy which enters the soil column can become trapped

as upgoing and downgoing wave-fields reflected successively from the free surface and the bedrock. Resonances occur at the surface due to constructive interference at frequencies given by multiples of one-quarter wavelength of the shear or compression waves in the soil column. The spectral amplitudes of the resonances are determined by the stiffness and density contrast at the rock/soil interface as well as details of the variation of shear modulus and soil density with depth.

The resonance phenomena and the general increase in amplitude with decreasing velocity of propagation are greatly affected by material damping. This causes the multiply-reflected energy to gradually damp out and can be observed as a successive decrease in spectral amplitudes at the higher modes (Silva et al., 1988).

It is important to point out that the near-surface amplification of ground motion may occur at rock sites as well. Physically, all that is required is a significant change in propagation velocity over a depth range of at least one-quarter wavelength (Joyner and Fumal, 1984). This effect may be responsible for a tendency of soft rock (sedimentary) to show some amplification relative to hard rock (granitic) (Campbell, 1985) if near surface velocity gradients are present.

In summary, the effects of surficial materials are dependent upon the dynamic material properties of the soil as well as the underlying rock, the soil depth, and wave frequency.

6.2.2 Variability in Site Response

Other factors which influence site effects and their variability arise from the contributions due to lateral changes in material properties as well as topographic effects. Non-vertical spatial variation in propagation path parameters can have significant effects upon both the amplitude of site response and its frequency dependence. Examples of these nonhomogeneous conditions include dipping structures such as an inclined soil/rock interface, lensing of soil layers, and basin or valley features. These propagation-path perturbations can scatter energy as well as change the modes of propagation both locally and over considerable distances. Wave scattering is the primary cause of incoherence of strong ground motion over rather

short distances (King and Tucker, 1982; Abrahamson, 1985). Acceleration at soft sites (shear-wave velocity less than about 1000 ft/sec) generally becomes incoherent for frequencies above about 5 Hz over distances of 300-600 feet and greater (Abrahamson, 1985).

Some or all of these nonhomogeneous factors are present at all sites to some degree in both soil and rock. Naturally the degree of perturbation varies with each site and with frequency. Neglecting structure effects, the variability encountered in observations of site response may largely be due to these lateral changes in dynamic material properties as well as topography. For example, the presence of a dipping soil/rock interface or topographic surface may make site response strongly dependent upon source azimuth and, to some extent, source depth. Examples of the presence of a dipping interface and topographic relief are the Gilroy #1 and #2 rock/soil pair in California (Appendix 6.B) and the Rocco and Forgaria-Carnino pair at Friuli respectively (Silva et al., 1988).

While the presence of these two-dimensional features is likely to some extent at most sites, in many cases the effects on ground motion predicted by the simple plane layered models and vertical propagating shear waves predominate over the frequency range of engineering interest (Seed and Idriss, 1969; Joyner et al., 1976, 1981; Silva et al., 1988; Idriss, 1990, 1991; Appendix 6.B). The simple plane-wave propagation model for the one-dimensional case consequently represents a useful analytical tool to approximate site effects.

6.2.3 Material Nonlinearities

It is well known from laboratory testing that soils exhibit pronounced nonlinear behavior under shear loading conditions (see Sections 7 and 8). Shear modulus decreases with increasing strain with an accompanying increase in material damping (Drnevich et al., 1966; Seed and Idriss, 1970; Hardin and Drnevich, 1972; Seed et al., 1984). If this observation is applicable to in-situ soil properties subject to earthquake loading, then site response calculations must accommodate these strain dependencies as material nonlinearities.

The strain dependence of soil modulus and material damping has been well documented in numerous laboratory studies for sands and clays (Drnevich et al., 1966; 1967; Seed and

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Idriss, 1970; Hardin and Drnevich, 1972; Silver and Seed, 1971; Seed et al., 1984) (see Section 8). Various parametric relationships have been proposed to determine values of maximum shear modulus (at small strain levels) and variations of shear modulus and material damping with strain (Hardin and Drnevich, 1971; Martin, 1975). Nonlinear soil models which have been primarily developed from laboratory test results and utilized in dynamic analyses, include the Ramberg-Osgood model (Streeter et al., 1974; Faccioli et al., 1973), an elasto-plastic model (Richart, 1975), Iwan-type model (Joyner and Chen, 1975; Dames & Moore and Science Applications, Inc., 1978; and Taylor and Larkin, 1978), the hyperbolic model (Hardin and Drnevich, 1972; Finn et al., 1975), the endochronic model (Day, 1979), and the Davidenkov model (Martin, 1975; Pyke, 1979).

Each of the nonlinear models mentioned has certain limitations and advantages in describing the response of soils to the type of loading produced by seismic disturbances. An effort has been made in some models to predict permanent deformations, while others have included pore water pressure build-up and dissipation. Strain dependency of material properties from laboratory data is universally observed. It is reproducible and becomes significant for high levels of earthquake loading, i.e., shearing strains $> 10^{-2}\%$

6.2.4 Implications for Site Response Calculations

Clearly if in situ soil is behaving as laboratory-derived models suggest, nonlinear effects must be incorporated into site response analyses. In general, strain dependence of modulus and damping would result in a nonlinear wave equation with the full finite strain terms (Biot, 1965; Murnaghan, 1937). In this case, superposition is invalid, the wave equation does not separate into P, SV, and SH fields, and Fourier analysis cannot be applied. The approach generally adopted is to assume infinitesimal displacements, utilize a linear wave equation, and confine the nonlinearity to the constitutive relation.

If a nonlinear constitutive relation is utilized, then velocities and damping become strain dependent. This can result in a wavefield and site response that depart significantly from linear one-dimensional theory. For high levels of motion strain dependent wave velocities would generally result in a lower characteristic site period. Strain-dependent damping would

generally reduce overall response at high strain levels. Depending upon site structure and input motion, these effects could be very large and must be incorporated into site response analyses (Appendix 6.B). Either nonlinear or equivalent-linear approaches may be used to accommodate nonlinear soil response and, as indicated in the introduction, Appendix 6.B shows a comparison of the two approaches to each other and to recorded motions.

6.2.5 *Field Evidence of Soil Nonlinearity*

It has long been recognized that there may be problems associated with laboratory testing of soil samples (Woods, 1968; Seed and Idriss, 1970). Accurate determination of shear moduli is complicated by the effects of sample disturbance. Additionally, resonant column and cyclic triaxial tests do not exactly simulate the dynamic stress paths caused by the passage of shear waves in the frequency range of several seconds to over 20 Hz. In situ measurements with stress waves representative of earthquake loading would eliminate these problems, but it is very difficult to induce large strains with controlled amplitudes in natural deposits.

To investigate this problem, two approaches have been utilized: 1) controlled sources employing large-strain-inducing generators; and 2) site response analysis utilizing earthquake or explosion data recorded by horizontal and vertical arrays. The controlled source approach, utilizing soil boring pressuremeters, torsional borehole devices, and even controlled explosions, has demonstrated soil nonlinearities (Dobry, 1991). The soil nonlinearity that was observed was either in strain dependent shear-wave velocity or load-deformation relations. Laboratory and field derived curves were in reasonable agreement. Japanese investigators (Tokimatsu and Midorikawa, 1981) have utilized the change in predominant period of site resonances (as observed in response spectra) for different levels of seismic excitation to infer strain dependent velocity changes. The modulus reduction curves obtained in this manner agree in substance with laboratory derived curves for the same sites. A recent analysis of transfer functions and amplification factors computed from strong motion data found evidence of nonlinear soil response (Silva et al., 1988; Silva et al., 1989; Chang et al., 1990; Silva et al., 1991; Su et al., 1992; Silva and Stark, 1992).

Other investigators, using simple qualitative comparisons of ground response patterns for

areas comprised of stiff and soft sites between small magnitude sources and intensity data from large local earthquakes (Borcherdt and Gibbs, 1976; Benites et al., 1985) have not observed departures from linear conditions. Shear-wave data from vertical arrays have been adequately modeled by linear theory up to 20%g (Shima, 1962; Seed and Idriss, 1970; Joyner et al., 1976; Johnson and Silva, 1981, Chang et al., 1986). The consistency of the transfer function between a soil site and a rock site has been examined for various levels of shaking (Murphy and Lahoud, 1969; Rogers et al., 1974; Hays et al., 1979; Joyner et al., 1981; Rogers et al., 1984; Tucker and King, 1984; Jarpe et al., 1988). In all of these analyses, the authors do not indicate significant evidence of nonlinear soil response. However, it should be emphasized that in a majority of the cases analyzed, average shear strains throughout the soil column were probably less than $10^{-2}\%$. At this strain level, the modulus reduction curve, such as shown in Figure 6-9, predicts a shear-wave velocity decrease of only about 8%, well within the variability of these analyses.

6.3 Site Response Model

Certainly the most satisfying approach to account for the effects of surficial materials on strong ground motion is empirical. Ideally amplification factors could be developed based entirely upon observation of strong ground motion. Past studies using data recorded on rock and on different classes of soil profiles, such as stiff soils and deep cohesionless soils, have demonstrated large differences in spectral amplification (S_a/a_{max}) and in spectral velocity due to the presence of the soils (Seed et al., 1976; Mohraz, 1976; Joyner and Fumal, 1984). While these studies are extremely useful in a general sense, the limited number and size of earthquakes and different types of profiles preclude relying directly upon empirical results. In particular, few data are available for very high levels of shaking and for a variety of site conditions. Also, few recording sites have detailed soil profiles for which reliable soil properties are available. Because of these limitations, some form of computational analysis is desirable and direct observations of soil response can then be used as calibrations and to provide a basis for assessing the reasonableness of the results of analytical computations.

The conventional computational approach in developing spectral amplification factors

appropriate for specific soil profiles would involve selection of suitable time histories to serve as control motions and a suitable computational formulation to transmit the motion through the profile.

6.3.1 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 introduced 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 an 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 and 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 become easily 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 linear analysis is

preserved. A truly nonlinear approach requires the specification of the shapes of hysteresis curves and their cyclic dependencies. 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.

The most significant advantage of the equivalent-linear formulation is the preservation of the superposition principle. For linear systems, this principle permits spectral decomposition and frequency-domain solutions. One can then make use of the propagator matrix solution scheme (Haskell, 1960; Schnabel et al., 1972; Silva, 1976) for very efficient frequency domain solutions of the wave equation. The superposition principle then permits a spectral recomposition of the wavefields (sum over frequencies) through an inverse Fourier or Laplace transform. A non-subtle result of this is that the deconvolution process, that of propagating the control motion down rather than up, results in a unique solution. That is, for a given motion at the surface, within an equivalent-linear framework there is only one base motion (solution). In reality of course, if the soils are behaving in a nonlinear fashion and have degraded, many different input motions at the base of the soil could have resulted in a similar surface response.

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 (Schnabel et al., 1972; Silva et al., 1988).

6.3.2 Computational Scheme

The computational scheme employed to compute the site response uses the stochastic model (Section 3) to generate the power spectral density and spectral acceleration of the rock or control motion. This motion or power spectrum is then 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, the 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. 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 acceleration.

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. Stable estimates of site response can then be computed by forming the ratio of spectral acceleration predicted at the surface of a soil profile to the spectral acceleration predicted for the control motion.

The procedure of generating the stochastic power spectrum (Section 3), computing the equivalent-linear layered-soil response, and estimating peak time domain values has been incorporated into a code termed RASCALS.

6.3.3 Generic Soil Profile

The generic median soil profile adopted for the site response analyses in this study is shown in Figure 6-3. This standard profile was chosen to be consistent with the generally stiff soils present under nuclear power plants located in the eastern and central United States and is intended to capture the behavior of soils ranging from gravels and sands to low plasticity sandy clays. The profile is based upon the sand-like profiles used by Bernreuter et al. (1985) from their review of FSARs and PSARs for nuclear plant sites and is similar to their generic profiles. It was decided not to include a separate suite of categories of constant shear-wave velocity with depth for sites with largely uniform profiles as their response is very similar to

that of the generic gradient model (Section 6.4.2.3). Other profiles with very soft near surface soils are not considered as such sites would probably not be of interest for a NPP.

The number of site categories adopted for analysis is five and they are shown in Figure 6-3. The depths and depth ranges of the five categories are shown in Table 6-2. The constant velocity profiles indicated in Figure 6-3 show the average (weighted harmonic mean) velocity of the standard gradient model for each category. As previously mentioned, response analyses performed with the gradient model and with the constant shear-wave velocity model produce very similar response results. As a result, only the gradient models are used in the development of the site amplification factors.

In order to accommodate variation in shear-wave velocity profiles for the range of central and eastern North American sites, the median profile velocities and profile depth (depth-to-rock) are randomized using the model presented in Appendix 6.A. The uncertainty model for velocity and depth randomization is based on a probabilistic analysis of over 350 profiles taken from sites predominately in the conterminous United States. The statistics then represent both western and eastern sites as too few profiles were available for eastern sites alone to provide meaningful statistics. In the analyses, 50 random profiles for each category are used to develop the median and one-sigma response characteristics of the categories.

The 50 randomized profiles are presented in Figure 6-4 through 6-8 for Categories 1-5. The median profile is shown as the dashed line on each figure (note the velocity scale changes with each category). The bedrock median velocity is 6000 ft/sec and represents an average (assuming log normal distribution) of bedrock velocities beneath soil profiles for sites east of latitude 105°W (37 sites). The corresponding value for sites west of latitude 105°W is about 3000 ft/sec. These values (particularly the eastern at 6000 ft/sec) may be biased somewhat low as the drilling, particularly at deep stiff profiles, may not have penetrated far enough into unweathered basement to obtain shear-wave velocities representative of good quality rock. For the generic profile considered here, the effects of a higher median velocity are small (Section 6.4.2.2) and a lower median bedrock velocity results in lower amplification factors so the issue is not considered significant in applications to generic amplification factors. The

uncertainty of the bedrock velocity (σ_m) is taken as 0.30. This represents the expected variability in good competent basement rock in the central, eastern and southeastern United States.

While the median bedrock velocity beneath the eastern sites is about 6000 ft/sec, this value does not correspond to the surface shear-wave velocities of the Midcontinent and Gulf Coast crustal structures (Section 5.2). The shear-wave velocity of the top 1 km thick layer of the Midcontinent structure is about 9300 ft/sec while the top 7 km of the Gulf Coast structure has a velocity of about 7600 ft/sec. Both of these values are above the median 6000 ft/sec suggesting that the assumption of a uniform Poisson's ratio of 0.25 (used in deriving the crustal shear-wave velocities from measured compression-wave profiles) may not be appropriate in the very shallow crust. To assess the effects of including a crustal velocity gradient to go from 6000 ft/sec at the surface to 11,550 ft/sec at 1 km in the Midcontinent structure (the velocity at 1 km) and to 7600 ft/sec at the same depth in the Gulf Coast crustal structure, amplification factors were computed for Categories 2 and 5 using control motions with and without a gradient. The results are nearly identical since the control motions are always adjusted by varying the distance to the same fixed acceleration level. The very nature then of the way the amplification factors are defined, relative to outcrop peak acceleration values makes them reasonably insensitive to crustal velocity gradients. Care should be exercised, however, in applying factors taken from observations in the western United States or computed with a median bedrock velocity of 3000 ft/sec to significantly higher velocity eastern crustal conditions. In this case the implied crustal velocity gradient might be great enough to significantly affect the soil amplification.

The depth variation in each category was constrained to the ranges in depth to which each category applies. For example, Category 1 has a median depth of 20 ft and is intended to be applicable to profiles with depths ranging from 10-30 ft (0-9 ft is considered rock). The randomized depths were therefore constrained between 10-30 ft. For random profiles which exceeded 20 ft, the median Category 1 depth, properties from Category 2 were used. For Category 5, appropriate for profiles ranging in depth from 401 ft to bottomless soil sites, the depth randomization was initially limited to 750 ft. An additional Category 6, with a median

depth of 1000 ft, was taken to apply for profiles of depths from 751 ft and beyond (randomized depths of 751 to 1250 ft). Very little difference was observed in the computed amplification factors for Categories 5 and 6 (median depths of 500 and 1000 ft) for frequencies around 1 Hz and above, so Category 6 was eliminated. To adequately sample the wider range in profile depth which Category 5 now represents, the random sampling was extended to between 401 and 1000 ft.

6.3.4 Modulus Reduction and Damping Curves

Figure 6-9 shows the modulus reduction and damping curves appropriate for the generic soil profile. The effects of confining pressure are reflected in the depth ranges shown, with less damping and higher $G/G_{\max}$ for a given strain level as the depth increases. The development of the generic curves is presented in Appendix 7.A.

In order to accommodate variability in the modulus reduction and damping curves on a generic basis, the curves were independently randomized about the values shown in Figure 6-9. A log normal distribution was assumed with a $\sigma_{\ln}$ of 0.35 at a cyclic shear strain of $3 \times 10^{-2}\%$ with upper and lower bounds of 2σ . The truncation was necessary to prevent modulus reduction or damping models which are not physically possible. The random curves are generated by sampling the transformed normal distribution with a $\sigma_{\ln}$ of 0.35, computing the change in normalized modulus reduction or percent damping at $3 \times 10^{-2}\%$ 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. Figures 6-10 through 6-15 show 20 of the 50 realizations for each of the six sets of curves. As the figures indicate, the suites of curves are smooth and are constrained to preserve the general shape of the median curves.

6.3.5 Specification of Control Motion

As previously discussed (Section 6.1), strong ground motion observed at rock sites in Eastern North America has different spectral properties than that typical of Western North America rock motions (Figure 6-2). The difference in spectral content shown in Figure 6-2 are

significant and indicate that, for the same level of peak acceleration, ENA spectral content is higher than that in the WNA for frequencies greater than approximately 10 Hz.

Other sources of data also indicate that ENA ground motion recorded at rock or very shallow soil sites is richer in high frequency energy relative to analogous WNA recordings. These include aftershocks of the 1982 Miramichi, New Brunswick event (Cranswick et al., 1985), the 1982 Enola, Arkansas swarm (Harr et al., 1984), and the 1986 Northeastern Ohio event (Wesson and Nicholson, 1986) and its aftershocks (USGS, 1986).

The trends shown in these data are consistent with the rock or stiff soil recordings of strong ground motion. They indicate a significant and reasonably consistent difference in high frequency spectral content between the representative rock motion in the WNA and ENA crust. This observation suggests that it is necessary to use ground motion representative of the ENA region to serve as control or input motion for computing site response effects in ENA.

Since the computational scheme used to compute the motions at the surface of the profiles does not require time histories for control motions (Section 6.3.1), the point-source stochastic ground motion model presented in Section 3 may be used to define the outcrop power spectral density. In order that the outcrop motions used to excite the soil profile are in general accord with the final engineering model presented in Section 9 for rock motions, the base case or mean source and propagation path parameters listed in Table 3-1 were used. The point source was assumed to have a depth of 10 km in the Midcontinent crustal structure. Since the control motions are computed for fixed peak acceleration levels and moment magnitudes (Table 6-3), the crustal structure was taken as a half-space. Figure 6-16 shows the suite of response spectra computed for the seven of the eight levels of control motions shown on Table 6-3 (0.05g is not shown in Figures). A low-pass filter (5-pole Butterworth) has been applied at 62.5 Hz to emulate the effects of processing and to bring the spectral levels to saturation at peak acceleration for frequencies near 100 Hz.

6.4 Development of Site Amplification Factors

Site amplification factors were computed as the ratio of 5% damping response spectral acceleration (S_a) computed at the surface of each site for each randomized profile to 5% damping response spectral acceleration (S_a) computed for the bedrock (outcrop) motion. In addition, both peak acceleration and peak particle velocity were computed for the site and outcrop as well. Levels of input motion (rock outcrop) of 0.05, 0.1, 0.2, 0.3, 0.5, 0.75, 1.0, and 1.25g were used to accommodate effects of material nonlinearity upon site response. Table 6-3 shows the magnitudes (M), distances (R), peak accelerations, and peak particle velocities of the outcrop motions. Also shown are the predominant frequencies, estimated from the random process theory, associated with the peak accelerations and peak particle velocities. Of interest is the much higher predominant frequency associated with peak acceleration than is generally exhibited for WNA motions. Interestingly, the predominant frequencies associated with peak particle velocities are generally close to those appropriate for WNA. The outcrop V/A (cm/sec/g) ratios display expected magnitude dependency and weak dependence upon distance.

6.4.1 Amplification Factors

The amplification factors, 5% damped S_a (soil)/ S_a (rock), were computed at approximately 90 frequencies from approximately 0.25 Hz to 50 Hz. The frequency range of interest is from 1 Hz to 34 Hz and more specifically at discrete frequencies of 1.0, 1.5, 2.5, 3.45, 5.0, 7.5, 10.0, 15.0, 25.0, and 34.0 Hz.

The entire set of factors is shown in Appendix 6.C, Figure Sets 6.C.1-6.C.5 for the five soil categories. As an example, Figures 6-17 to 6-19 show the factors and sigma values computed for Category 3 (120 ft) with control motions of 0.10, 0.50, and 1.00g. Due to the randomizing over depth, only a minor contribution to the fundamental resonance is present. This is a desirable feature for generic amplification factors and closely resembles recently derived empirical Western North America amplification factors (Section 6.5).

Table 6-4 summarizes the amplification factors for the five categories and eight input acceleration levels. The largest uncertainty (σ_m) is about 0.4 (Table 6-4) and occurs near the fundamental resonance of the median profile where the soil column is, on average, having

its greatest amplification effects (Toro et al., 1992). The effects of damping are seen in the decrease in amplitude with increasing frequency. The reversal in this trend near 50 Hz occurs where the rock outcrop acceleration response spectra (Figure 6-16) begin to saturate to peak acceleration. At 100 Hz the amplification factors are then essentially peak acceleration ratios.

Subtle effects of nonlinear response are seen in Figure 6-18 in the flatter slope and more pronounced resonance shown in the 16th percentile factor compared to the 84th percentile. For the higher motions, the overall damping is greater possibly due to higher bedrock velocities and softer profiles than median values. For the higher levels of control motions, 0.50g in Figure 6-18 and 1.00g in Figure 6-19, the nonlinear effects are dramatic. There is a decrease in the maximum spectral amplification and an increasing fall-off with frequency. Also, the largest uncertainties, for the 1.00g control motions (Figure 6-19), are not at the median resonance but at very high frequencies (30-40 Hz). For very high levels of motion, the predominant effect of the soil column is attenuation through damping rather than amplification through the velocity gradient. An important inference from these results is that for high levels of motion, accurate prediction of site response depends more strongly on an accurate model of the dependency of material damping and shear modulus on shear strain than on the shear-wave velocity profile. This result is substantiated in the comparisons of site response analyses to recorded motions in Appendix 6.B.

For the 1.0g control motions, the computed amplification factors show a sharp decrease with increasing frequency and are less than one above about 8 Hz. The minimum value is about 0.5, close to the empirical minimum of 0.6 (Section 6.5). For the largest control motions, 1.25g, the maximum cyclic shear strains average around 0.5%.

A convenient visual presentation of the amplification factors is shown in Figures 6-20 to 6-27. Each figure shows the factors plotted versus median profile depth for five levels of control motion: 0.10, 0.30, 0.50, 0.75, and 1.00g. Figures 6-20 to 6-25 are for oscillator frequencies 1.0, 2.5, 5.0, 10.0, 25.0, and 34.0 Hz. Figures 6-26 and 6-27 show predicted amplifications for peak accelerations and peak particle velocities respectively.

The spectral amplification figures, in a general sense, show a frequency-dependent resonance versus depth. The amplification factors generally increase with depth at 1.0 Hz and show a peak at 2.5 Hz at a depth near 120 feet. The peak shifts as expected, and is located near a soil depth of 50 feet for a frequency of 5.0 Hz. At 10.0 Hz, the trend shown by the 1 Hz calculations has reversed and the peak is occurring between the surface and 20 feet. The peak acceleration amplification factors (Figure 6-26) follow the trend of the 10.0 and 25.0 Hz response spectral amplification factors (Figures 6-22 and 6-24). Acceleration factors for Western North America, due to lower frequency content, might be expected to follow more closely the trend shown by the 2.5 and 5.0 Hz response spectral amplification factors. Peak particle velocity (Figure 6-27) is in general agreement with the trends shown by a combination of the 1.0 and 2.5 Hz response spectral amplification factors. This is in accord with the lower predominant frequencies associated with the input (rock outcrop) particle velocities (Table 6-3) compared to the corresponding accelerations. The predominant frequencies at the surface of the profiles for accelerations and particle velocities is approximately 10-20 Hz and 1-3 Hz respectively, and is naturally site dependent.

The effects of nonlinear soil response are easily seen in the range in amplification for different input motions. The effect appears to be both frequency and soil-depth dependent. For low frequencies (1.0 and 2.5 Hz) and shallow profiles (< 120 ft) the results do not depend strongly upon input acceleration level. As the profile deepens and as frequency increases, the dependence upon input acceleration level increases substantially. However, it should be kept in mind that the results span an order of magnitude in input acceleration levels (0.1-1.0g) which would correspond to a change of slightly more than three magnitude units at a fixed distance (assuming $\log(A_p) = 0.3 M$; Boore, 1983). Even with this range in motions, the maximum average variation of the amplification factors is approximately 50%.

The general trend of decreasing amplification with increasing levels of input motion is shown consistently with peak acceleration, peak particle velocity, and response spectral amplification at high frequency (10.0, 25.0, 34.0 Hz) and may be a consequence of increased damping at higher overall shear-strain levels. At lower frequencies, however, there are some reversals to this trend. This may be associated with a shifting and an accompanying increase in

amplitude of resonances along with an increase in shear-wave amplification due to a greater velocity gradient for higher levels of input motion. In some cases these effects may predominate over increased damping at lower frequencies. As described earlier, the net amplification is a trade-off of counteracting effects between soil damping, resonance effects, and an increase in shear-wave amplitude due to velocity gradients.

The amplification factors for the deepest profiles at high frequencies are predicted to be significantly less than one for higher levels of input motion. This can be attributed to the effects of damping and may be exaggerated. In order to adjust the predicted amplifications for these possible effects in a conservative manner, a lower bound of 0.6 is adopted for the set of computed amplification factors. This value is in agreement with the lower bound in the empirical factors for Western North America (Section 6.5, Figure 6-38).

The 5% damped response spectral amplification factors together with those predicted for peak acceleration and peak particle velocity are summarized in Table 6-4 for each level of input (rock outcrop) acceleration. The entries with asterisks are those which are lower than the adopted minimum of 0.6. In these cases the value of 0.6 should be substituted for use. To accommodate levels of input motion between entries on the table, simple linear interpolation is recommended.

It should be reiterated that these amplification factors approximately account for the effects of surficial geology by generic soil category. Although detailed site specific results could be different from those predicted for these generalized categories, we believe that the results presented in Table 6-4 are appropriate and represent a state-of-the-art assessment for use in seismic hazard estimation for the Central and Eastern United States.

6.4.2 Consideration of Special Cases

In the development of the amplification factors there are issues of importance regarding the effects of a number of parameters. In particular, the effects of control motion magnitude, bedrock velocity, and nature of the profile gradient should be addressed. In the following sections, the effects of these issues on the computed amplification factors will be quantified

by making comparisons to the amplification factors computed using base case parameters with results using different magnitudes, median bedrock velocities, and uniform profiles rather than profiles with a velocity gradient.

6.4.2.1 Effects of Control Motion Magnitude. For the same peak acceleration value, the magnitude of the control motions can be changed with a change in distance, all other parameters remaining constant. The change in magnitude results in a change in corner frequency which primarily affects the low frequencies and, consequently, peak particle velocity. Since shear strain levels have a dependence on particle velocity, a change in magnitude, while keeping the peak acceleration constant by changing the distance, could result in a change in the computed amplification factors.

To evaluate the size of these effects, amplification factors for Category 2 (50 ft) with control motions of 0.30 and 0.50g were computed using **M** 7.5 instead of **M** 6.0 (0.30g) and **M** 6.5 (0.50g) (Table 6-3) for the control motions. The results are shown in Figures 6-28 and 6-29 for the two levels of control motions. The solid lines represent the amplification factors computed with the base case magnitudes (**M** 6.0 and **M** 6.5) while the dotted lines are for the **M** 7.5 control motions. The effects of the greater low-frequency energy (larger outcrop peak particle velocities) associated with the larger magnitude is to build up slightly higher strain levels resulting in a small decrease in the amplification factors. The effect is small but it does illustrate the effect of frequency content on nonlinear soil response and suggests caution in simply scaling recorded motions for site response analyses.

6.4.2.2 Effects of Bedrock Velocity. The impedance contrast across the soil-rock interface is a controlling factor of how much energy is transmitted to the profile as well as how much is reflected back into the profile. To examine the sensitivity of the amplification factors to the median bedrock velocity, amplification factors for Categories 2 and 5 were computed using median bedrock velocities of 3000 ft/sec and 9000 ft/sec. The results show a slight increase in the factors using the median bedrock velocity of 9000 ft/sec and a substantial decrease with the 3000 ft/sec median bedrock velocity. Figures 6-30 and 6-31 for Category 5 using a control motion of 0.50g illustrate the general results. Figure 6-30 shows the

amplification factors using both the base case median bedrock velocity of 6000 ft/sec (solid line), and the velocity of 9000 ft/sec (dotted lines). The factors computed using the higher bedrock velocity are slightly larger at low frequencies and about the same at high frequencies. Similar degrees of differences are present in the other categories with the maximum departure being about 10% for Category 1.

Using a median bedrock velocity of 3000 ft/sec, compared to 6000 ft/sec results in a significant difference in amplification factors. This represents a factor of two reduction in the impedance contrast and reduces the motions a maximum of about 30% near the profile resonance and, due to the nonlinear soil response, progressively less at higher frequencies. This trend is illustrated in Figure 6-31 which shows median amplification factors computed for Category 5, with a control motion of 0.50g using both 6000 ft/sec and 3000 ft/sec median bedrock velocities. The lower velocity results show a slightly lower frequency resonance with lower amplitude. As frequency increases, the difference in the two factors decreases.

The effects of median basement shear-wave velocity are not large for a 50% increase but do result in slightly greater (maximum about 10%) factors near the profile resonance. On the other hand, a 100% reduction in the median bedrock velocity, from 6000 ft/sec to 3000 ft/sec, results in an overall reduction in the amplification factors by about 30% near the profile resonance and progressively less for higher frequencies.

6.4.2.3 Effects of Uniform Velocity Profiles. While most shear-wave velocity profiles show a general increase in velocity with depth, generally most rapid near the surface, there are profiles which increase only slightly in velocity with increasing depth. Generally larger amplification is expected from profiles with strong velocity gradients as opposed to more uniform profiles, with all other factors remaining the same. As a result, it is believed that the amplification factors computed using the generic profile, which contains a strong velocity gradient, particularly in the upper 100 ft (Figure 6-3), are not unconservative in applications to generally uniform profiles.

To quantify this, amplification factors were computed for Categories 3 (120 ft) and 4 (250

ft) profiles using a constant shear-wave velocity of 1500 ft/sec, near the average velocity of Category 4 (about 1800 ft/sec, Figure 6-3). The results are compared to the gradient model in Figures 6-32 and 6-33 for a control motion of 0.50g. The amplification factors computed using the gradient model (base case) exceed those of the uniform profile except near 1 Hz for Category 3 profiles. The uniform profile factors are slightly larger near 1 Hz because its median resonance occurs at a lower frequency than that of the gradient model. In general, the gradient model results in higher factors than the uniform profile, particularly if the median resonances are at about the same frequency, as is the case with Category 4.

6.5 Comparison of Computed Amplification Factors to Empirical and other Analytical Results

Direct comparison of the results presented here with other results for 5% damped response spectral amplification factors, as well as for peak acceleration and peak particle velocity, is complicated by several factors. Comparisons with empirical observations at well-defined soil sites appropriate for Eastern North America motions are not possible. Few, if any, recordings exist made at soil/rock pairs with soil sites representative of our classifications and at close distances to large ($m_b > 5$) events. Also not enough data exist, particularly at close distances, at both rock and soil sites to sufficiently constrain empirical attenuation relationships (Section 2). Recordings of aftershocks of Eastern North America events (Boatwright and Astrue, 1983; Mueller and Cranswick, 1985; Cranswick et al., 1985) show rather large amplifications at high frequencies for very shallow soil sites or fractured rock sites relative to hard-rock sites.

Comparisons with Western North America observations are complicated by differences in underlying rock shear-wave velocities (Section 6.4.2.2) as well as differences in high frequency spectral content of the control motions (Section 6.4.2.1).

An additional complicating factor arises in comparisons between response spectral ratios and Fourier spectral ratios (Silva et al., 1988). Caution must be exercised in making inferences about amplification factors appropriate for 5% damped response spectra based upon analyses performed with Fourier amplitude spectral densities. Silva et al.(1988) show that the

difference between response spectra and Fourier amplitude spectra in computing the effects of site response is frequency and site dependent, and can vary with magnitude as well, even for linear site response.

In addition to the differences between response and Fourier spectral amplifications resulting from analytical procedures, observationally this difference will likely be greater. This arises because the Fourier spectrum for a transient event must be smoothed over neighboring frequencies to produce statistically stable estimates. This procedure degrades resolution but must necessarily be performed to reduce the variance of the estimates prior to taking ratios. If the proper smoothing is not performed, statistical fluctuations in the Fourier spectra may seriously bias the ratios. As a consequence of the necessity to smooth, the estimates of site response based upon Fourier amplitude spectra computed from observations may be directly related to the degree and nature of the smoothing process. Because of this, comparisons between soil site effects based upon Fourier amplitude spectra computed from observations to computations using response spectra are unclear and will not be pursued.

6.5.1 Comparisons To Empirical Results

Although direct comparisons of the amplification factors presented in Table 6-4 for ENA with other results is not strictly appropriate due to differences in bedrock velocities and control motion spectral content, similarities should exist both in sizes of factors and in trends. Comparisons with empirical results can then provide a basis from which to judge the reasonableness of the computed values.

In order to provide a basis for comparison to empirical amplification factors, regression analyses for peak acceleration and 5% damped response spectra were performed on a set of reprocessed strong motion data (Appendix 2.C). The data set represents available unprocessed strong motion records from earthquakes in active tectonic regions with M above about 5.8. The data were reprocessed on an individual record basis to extend both the high and low frequency limits (Silva and Abrahamson, 1992). The reprocessed data set then represents an extensive set of uniformly processed records with an optimal useable bandwidth for each record.

In the regression analyses a functional form was selected which directly evaluates the soil-to-rock amplification factors. The functional form of the ratio was taken to allow for a nonlinear relationship between the motion on soil and that on rock (Equation 2C-1 in Appendix 2.C). The results for both deep and shallow soil show a clear dependence of the amplification factors on magnitude and distance, or equivalently on rock peak acceleration level. For a given magnitude, the amplification factors increase with increasing distance. The soil motions increase with increasing distance at a greater rate than the corresponding rock motions. The trend is identical to that shown in the computed amplification factors and is due to the effects of nonlinear soil response.

In order to provide a direct comparison between the computed amplification factors for Eastern North America and the empirical Western North America factors at different rock acceleration levels, empirical motions were calculated at rupture distances of 1, 5, 10, 25, and 50 km for rock, deep soil, and shallow soil for M 6.5 (majority of the data) (Appendix 2.C). The corresponding rock or "control" motion peak acceleration values are 0.06, 0.12, 0.26, 0.42, and 0.75g. The closest outcrop control motion values used in computing the amplification factors are 0.05, 0.10, 0.30, 0.50, and 0.75g (Table 6.3). Figures 6-34 to 6-38 show the comparisons of the computed amplification factors and the corresponding empirical factors for the five outcrop peak acceleration values. Only deep soil is used in the comparison because these sites represent the bulk of the data and therefore provide better constrained regression results. To provide soil/rock conditions closer to Western North America, the computed amplification factors are shown with median bedrock velocities of both 3000 ft/sec and 6000 ft/sec.

For the smallest control motions, about 0.05g, Figure 6-34 shows similar levels of amplification for the empirical and theoretical factors, particularly for the results using the median bedrock velocity of 3000 ft/sec. Interestingly, both the analytical and empirical results peak near 1 Hz with the empirical showing a broader peak possibly due to a wider sampling in depth range. For these low levels of motion the empirical and analytical factors are greater than one for all frequencies.

The next higher level of control or rock motions, about 0.10g, are shown in Figure 6-35. Both the empirical and analytical amplification factors are reduced at higher frequencies with the empirical showing the greatest decrease. This result, expected at higher levels of motion, is surprising and suggests that the effects of nonlinear soil response can extend down to low levels of control motions, even into the range of 5-10%g. The analytical results do not show nearly as great a decrease, possibly because the median profile is comparatively stiff. Figure 6-36 shows the factors for about 0.30g rock motions and both the empirical and analytical factors show a further decrease. The analytical factors are less than the empirical above about 20 Hz which may reflect higher soil damping due to the larger high frequency energy content in the Eastern North America control motions (Section 6.2). The minimum in the empirical factors occurs near 20 Hz, unlike in the analytical where it occurs near 50 Hz and is much sharper as well. As previously indicated, this feature in the analytical factors is due to the effects of kappa (0.006 sec) and the low-pass filter at 62.5 Hz. The soil spectra are nearly flat in this range due to the damping (the minima are more pronounced at higher levels of motion) while the rock spectra are falling off to peak acceleration levels with increasing frequency (Figure 6-16). This trend is shifted to lower frequencies for Western North America motions due to the higher average kappa values (0.04 sec, Silva and Darragh, 1993). The effect is also less pronounced for the western motions because the effects of the high kappa values gradually reduces the high frequency energy to low levels before the antialias filters are applied.

The two highest levels of motions, about 0.50 and 0.75g, are shown in Figures 6-37 and 6-38. While some data exist for the 0.50g results, the 0.75g amplification factors are largely extrapolations. As with the lower levels of motions, the trend in the empirical is for decreasing amplification with increasing levels of motion. The analytical factors exceed the empirical at low frequencies and are smaller above 20 Hz for 0.50g and about 10 Hz for 0.75g control motions.

The results of these comparisons are very encouraging. The analytical amplification factors show very similar shapes and trends to the empirical. Both are generally smooth and for the lowest motions, show peaks near 1 Hz, and are greater than one for all frequencies. At

higher levels of motion, both the empirical and analytical factors are reduced in amplitude with the empirical showing a larger dependence at the lower levels of motions (0.05 to 0.10g). The analytical factors are generally greater than the empirical at low frequency, even for the bedrock velocity of 3000 ft/sec. This is possibly due to the relatively stiff median shear-wave velocity profile. At higher frequencies and higher levels of motion, the analytical factors are less than the empirical which may be a result of a greater amount of high frequency energy in the eastern control motions.

These comparisons are very favorable and suggest that the combination of 1) the vertically propagating shear-wave model, 2) generic modulus reduction and damping curves based on recent high quality laboratory testing and reflecting the effects of confining pressure, and 3) the equivalent-linear methodology represents an accurate analytical tool in estimating the effects of soils on strong ground motions.

6.5.2 Comparisons To Analytical Results

Comparisons with results based upon computations using similar methodologies can also be useful to assess the reasonableness of the computational procedures. Both types of comparisons are shown in Table 6-5.

The Lawrence Livermore National Laboratory entries (Bernreuter et al., 1985) are average results of response spectral ratios computed from SHAKE analyses using predominately Western North America outcrop motions with Central and Eastern North American generic soil profiles. In the Lawrence Livermore Laboratory analyses, the median damping has been fixed at 7% and is noted in the table. Alongside the Lawrence Livermore Laboratory entries are the results of the present study (Table 6-4) for an input acceleration level of 0.75g, which corresponds to an average shear-wave damping of approximately 7%. While the site categories and ranges of the two studies are not identical, considerable overlap does exist. A comparison of the results from the analytical studies shows very similar results for low frequencies with the difference increasing with increasing frequency. This would be expected considering the different high frequency spectral content of the outcrop motions used in the two studies. However, even at the higher frequencies the trends with profile depth are

similar. At higher frequencies, the factors computed using outcrop motions with Eastern North America spectral content decrease more rapidly with soil depth than those computed using control motions with correspondingly less energy. These trends are similar to the Western North America empirical results and are likely due to the lower frequency western outcrop motions. Typical rock motions for tectonically active regions generally have substantially reduced energy, relative to corresponding eastern motions for frequencies above about 10 Hz for the same level of peak acceleration (Figure 6-2). As a result, damping may be depleting the higher frequencies to a relatively greater degree using eastern input motions.

Boore and Joyner (1991) have recently developed Fourier amplitude amplification factors for deep stiff (200 ft and beyond, UBC Category S2) Central and Eastern North America sites. The factors result from a linear analysis using a profile similar to Category 5 and extended to depth of about 2000 ft. The average damping in the profile is approximately 2%. While it is difficult to directly compare Fourier amplification factors to 5% damped response spectral amplification general agreement should exist for the common ranges in profiles. Figures 6-39 and 6-40 show the Boore and Joyner Fourier amplification factors compared to the response spectral factors for all five categories at two levels of control motion, 0.10 and 0.50g. For a rock outcrop motion of 0.10g, the average damping for Category 5 (randomized depth 401-1000 ft) is about 1-2% and 3-4% for 0.50g control motions. From the figures, it is apparent that the results are in close agreement recognizing that the Fourier amplification factors are applicable to sites ranging from about 200 ft to bottomless profiles. At 0.10g, Figure 6-39, the amplification factors for each category form a family of curves beginning at low frequency with Category 5. The approximately constant levels are generally consistent with those of Boore and Joyner out to about 5 Hz where the Fourier amplification curve begins to fall off more rapidly. Beyond that the response spectrum factors exceed the Fourier factors.

At 0.50g input motion, Figure 6-40, the response spectra factors show a reduced amplitude particularly in the higher frequencies, reflecting the effects of nonlinear soil response, a trend similar to that shown in the empirical amplification factors for Western North America (Figure 6-37). Due to the assumptions of linearity, the Fourier amplification factors are

independent of control motion amplitude. At these high levels of motion, an average of Categories 3, 4, and 5 (120 ft to 500 ft) amplification factors would be quite similar to the linear factors from about 1 Hz to about 5 Hz and lower beyond. Adding in the 0.10g factors would likely result in a similar set of factors over the range of 1-20 Hz. Considering that the linear Fourier amplification factors were computed using a similar velocity profile but extended to about 2000 ft and completely different assumptions concerning material damping (constrained to observations for the Mississippi Embayment, Boore and Joyner, 1991) the agreement between the two is quite good and suggest that a large uncertainty does not exist in estimating the effects of site condition on strong ground motions.

6.6 Assessment of Vertical Motions

This section treats the specification of the vertical component of strong ground motions. First a summary is presented of the current approach to specifying the vertical component of strong ground motions followed by development of a preliminary computational model based on the stochastic ground motion model (Section 3). Model predictions are then compared to empirical Western North America vertical-to-horizontal (V/H) ratios. Finally, guidelines are suggested for specifying vertical component motions for Midcontinent and Gulf Coast rock sites as well as the five soil site categories.

6.6.1 Current Approach

The current procedures for the selection of the vertical component of earthquake ground motions include those that are used for buildings and industrial facilities (other than nuclear plants) and those that are specified for nuclear plants in documents such as Reg Guide 1.60.

6.6.1.1 Buildings and Industrial Facilities: Typically, the vertical component is seldom used for these structures. When it is used in the analysis, the spectral ordinates of the vertical component are considered to be 2/3 those of the spectral ordinates of the horizontal component. For some critical structures, and when the structure is within a short distance of the source, the spectrum of the vertical component is selected to be equal to or slightly larger than the horizontal spectrum for frequencies greater than about 2 Hz. In many instances, a set of motions (two horizontal and one vertical component), that had been

recorded at close distances to the source, is used in this case. Because of the scarcity of such recorded data, however, it is not possible to utilize this procedure at many sites.

Nuclear Plants: The most widely used procedure to specify the vertical component for nuclear plants has been the procedure included in Reg Guide 1.60. The spectral ordinates for the vertical component are set equal to those of the horizontal component at high frequencies (greater than about 10 Hz) and in the "acceleration" region (typically ranging from about 2 to about 10 Hz) of the spectrum. In the velocity and displacement regions (ie. for frequencies less than about 2 or 3 Hz), the spectral ordinates of the vertical component are set equal to 2/3 those of the horizontal component.

For some recent analyses for nuclear plants, the spectral ordinates at a nearby rock outcrop were selected to be 2/3 those of the horizontal spectral values for the entire frequency range. This vertical rock outcrop motion was then used as input rock outcrop motion to a soil column to calculate the vertical motion at the soil site.

In the Standard Review Plan (NUREG-0800) or Reg Guide 1.60, neither specific recommendations are made nor are guidelines presented regarding applications to eastern sites. As a result, even with the reservation that the computational model presented in the next section is preliminary, it is felt beneficial to present recommendations for vertical-to-horizontal ratios which specifically consider the differences in typically western and eastern strong ground motions.

6.6.2 Computational Model

To model vertical motions, incident inclined compression waves from the stochastic ground motion model (Equation 3-1) are assumed incident at the top of the source layer and the P-SV propagators of Silva (1976) are used to model the crust and soil response to inclined P-SV wavefields. The angle of incidence at the top of the source layer is computed by two-point ray tracing through the crust and soil column (if present) assuming incident compression waves.

To model soil response, a soil column is placed on top of the crustal structure and the incident inclined P-SV wavefield is propagated to the surface where the vertical (or radial) motions are computed.

At close distances, for typical crustal structures without strong near-surface compression-wave velocity gradients, the predominant motion on the vertical component is principally due to the SV wavefield. In a soil column (particularly deep profiles) however, because there is usually a large compression-wave velocity gradient, larger for compression waves than for shear waves as Poisson's ratio generally decreases with depth, the vertical component is usually controlled by the compression wavefields. The separation of rock and soil sites in terms of predominant wavefields in the vertical component depends on specific velocity profiles (soil as well as crustal), source depth and mechanism through their effect on incidence angles, as well as the depth of the water table.

The water table may be a significant factor at soil and perhaps soft rock sites because a significant increase in the compression-wave velocity to values near the fluid phase (about 5000 ft/sec) is generally encountered. Typically at soil and soft rock sites, beginning at the surface, a velocity gradient exists for both compression and shear waves with a Poisson's ratio in the range of about 0.3 to 0.4. Poisson's ratio may decrease with depth slightly until the water table is reached where it jumps to values approaching 0.5. At this point, the fluid phase controls the compression-wave velocity until a depth is reached where the compression-wave velocity of the skeleton material exceeds the fluid phase velocity. From this point on, Poisson's ratio generally shows an overall slow decrease with depth.

This trend is clearly seen in the profile of Poisson's ratio shown in Figure 6-41. The Poisson's ratio is based on velocities measured with a suspension log tool in a deep hole at the Savannah River site. The water table is located at a depth approximately where the log begins (near 40 ft) and shows a general decrease with increasing depth to about 500 ft. Below 500 ft, the profile becomes more erratic as a different formation is encountered. However, even with these fluctuations, the overall trend is for Poisson's ratio to decrease as the skeleton compression-wave velocity increases. Naturally, site specific factors influence

the picture but this overall trend is a generally common feature of soil and soft rock sites.

On a generic basis, the most straightforward approach to model the vertical motions is to assume that the water table is at the surface. The effects of placing the water table at some depth below the surface will depend on the exact depth, the compression-wave damping above the water table, and, to some degree, on source distance and depth, and may result in increased motions at high frequencies. Since these sensitivities are not yet resolved, the current analyses will assume the water table is at the surface and concentrate on general trends in the computed vertical-to-horizontal response spectral ratios. Conservative recommendations will then be given as a guide in assessing vertical motions. Site specific analyses should be conducted with ranges of compression- and shear-wave velocities and water table depths based on measured values.

6.6.2.1 Treatment of Soil Response For Vertical Motions. Commonly equivalent-linear site response analyses for vertical motions have used strain iterated shear moduli from a horizontal motion analysis to adjust the compression-wave velocities assuming either a strain independent Poisson's ratio or bulk modulus. Some fraction (generally 100% to 30%) of the strain iterated shear-wave damping is used to model the compression-wave damping and a linear analyses performed for vertically propagating compression waves using the horizontal control motions scaled by 2/3 (I.M. Idriss, 1993, personal communication).

The equivalent-linear approach implicitly assumes some coupling between horizontal and vertical motions. This is necessitated by the lack of well determined degradation and damping curves for the constrained modulus. Ideally, the strain dependency of the constrained modulus should be determined independently of the shear modulus. Also, the approach assumes vertically-propagating compression waves and not inclined P-SV waves. Additionally, the use of some fraction of the horizontal control motion is an approximation and does not reflect the generally greater high-frequency content of vertical component motions at rock sites due to lower kappa values (Section 6.6.2.2, Anderson, 1991).

Alternatively, fully nonlinear analyses can be made using two- or three-component control

motions (Costantino, 1967; 1969; Li et al., 1992). These nonlinear analyses require two- or three-dimensional soil models which describe plastic flow and yielding and the accompanying volume changes as well as coupling between vertical and horizontal motions through Poisson's effect. While these analyses are important to examine expected dependencies of computed motions on material properties and may have applications in studies of soil compaction, deformation, slope stability, and component coupling, the models are very sophisticated and require specification of many parameters, at least some of which are poorly understood.

In the current implementation of the equivalent-linear approach in estimating vertical to horizontal response spectral ratios, the horizontal component analyses are performed for vertically propagating shear waves using the RVT methodology (Section 6.2.1). To compute the vertical motions, a linear analysis is performed for incident inclined P-SV waves using low-strain, compression- and shear-wave velocities derived from the generic shear-wave velocity profiles and an assumed Poisson's ratio (Section 6.6.3). The water table is taken at the surface and compression-wave damping is assumed to be equal to the low strain shear-wave damping (Johnson and Silva, 1981). The horizontal component and vertical component analyses are assumed to be independent.

These approximations, linear analysis for the vertical component and uncoupled vertical and horizontal components, have been checked by comparing results of fully nonlinear analyses at reference sites Gilroy #2 and Treasure Island to recorded vertical and horizontal motions from the 1989 Loma Prieta earthquake (Appendix 6.D). The nonlinear analyses indicate that little coupling exists between the vertical and horizontal motions for the ranges in control motions analyzed (maximum 0.473g at Gilroy #1, Appendix 6.B). Since the median generic profiles, Categories 1-5, are much stiffer than the Gilroy #2 profile (Appendix 6.B), resulting in significantly lower strains for the same control motions, the conclusions of Appendix 6.D are assumed to apply to the higher levels of control motions used in computing the amplification factors (Table 6-3). These assumptions are expected to result in conservative estimates of vertical motions since a higher degree of coupling implies degradation of constrained modulus and an accompanying increase in compression-wave damping.

6.6.2.2 Model Validation. The computational model is validated by modeling vertical component response spectra from the 1989 M 6.9 Loma Prieta earthquake at 22 rock and 2 soil sites (Gilroy #2 and Treasure Island). The fault distance range (surface projection of the top of the rupture surface) is from 1 to 80 km. Site specific kappa values for the vertical components at the rock sites are based on template fits of response spectral shapes computed assuming a $Q(f)$ of $150 f^{0.6}$ and a moment magnitude of M 6.9. The kappa values average about half that obtained using a similar procedure applied to the horizontal components (Silva and Stark, 1992; Silva and Darragh, 1993). These results are consistent with Anderson (1991) who found vertical component kappa values about half that of the horizontal components at rock sites. A summary of the sites, distances (hypocentral), and kappa values is given in Table 6-6. The compression and shear-wave soil velocity profiles are shown in Appendix 6.B.

In the simulations, the point source is located at a depth of 12 km (Silva and Stark, 1992) and the crustal model of Wald et al. (1991) is used. A stress drop of 200 bars is assumed which results in a source duration of about 6-7 sec, appropriate for the bi-lateral rupture of this earthquake (Wald et al., 1991; Silva and Stark, 1992).

Results of the modeling are shown in Figure 6-42 where the empirical and computed vertical component 5% damped response spectra are shown for each site. The model appears to capture the level and overall shape of the response spectra at most sites reasonably well. For the sites with distances of about 20 km, the incidence angle at the layer above the source is around 60° ; for more distant sites, a greater angle of incidence results in a general underprediction of motions which becomes progressively worse as distance increases. As a result, an upper-bound incidence angle is set at 60° . Apparently too little upward propagating energy from the point source is entering the structure. One could partition the energy into upward and downward propagating fields from the source (Harkrider, 1981) based on focal mechanism and treat the wave propagation in a more computationally rigorous manner. However, these results indicate that such a computationally demanding approach may not be warranted and suggest that the vertical motions possess a large stochastic component which is captured reasonably well by a physically simple model. Additionally, since the primary

interest in the vertical component motions is at close distances (< 20 km) where it can equal or exceed the horizontals at some frequencies (Section 6.6.4), the 60° upper bound is not of major practical concern.

A quantification of the goodness-of-fit to the vertical motions is shown in the uncertainty, Figure 6-43, and in the computed bias, Figure 6-44. The uncertainty plot indicates low levels of misfit particularly for frequencies exceeding about 6-7 Hz. Overall, the simple model appears to produce levels of uncertainty comparable to the horizontal component (Section 3.0). However, it should be kept in mind that this validation exercise is for one earthquake only and should be applied to a number of earthquakes and many more sites before the model can be considered carefully validated.

The model bias, Figure 6-44, indicates generally unbiased estimates except for frequencies around 4 Hz. This negative bias represents an overprediction and results from a resonance in the Wald et al. crustal model that is clearly seen in the plots of response spectra (Figure 6-42). If the velocity structure had been randomized in a manner similar to that done with the soil profiles and a median response spectrum computed for each site, it is likely that the bias would be near zero throughout most of the frequency range.

6.6.3 Velocity Models

6.6.3.1 Crustal Models. To compute outcrop vertical-to-horizontal response spectral ratios, the two standard crustal models are used: Midcontinent and Gulf Coast (Section 5). The shear- and compression-wave velocities for these two regions are shown in Figure 6-45. The compression-wave velocities represent measured values based on a number of long range refraction studies while the shear-wave models are derived from the compression-wave velocities assuming a constant Poisson's ratio of 0.25.

6.6.3.2 Soil Models. The shear-wave velocity model is based on the median profiles of the five site categories shown in Figure 6-3. The corresponding compression-wave velocities are computed by assuming a Poisson's ratio which decreases linearly with depth. The intercept

and slope are based on the Savannah River suspension log results (Figure 6-41). The Savannah River data represents the best constrained compression- and shear-wave velocity measurements for deep profiles in the profile database. The equation for Poisson's ratio is given by

$$\sigma(z) = 0.48 - 2.6 \times 10^{-4}z \quad 6-1$$

where $\sigma(z)$ is Poisson's ratio and z is depth in meters. This equation assumes the water table extends to the surface. The bedrock compression-wave velocity is computed by assuming a Poisson's ratio of 0.25.

In the site response analyses for the vertical component, the small-strain shear-wave damping is used for the compression-wave damping. These values are around 1% and decrease slightly with depth. The values are listed in Table 6-7. As indicated previously, linear analyses are performed for the vertical components.

6.6.4 Development of V/H Ratios

In computing the horizontal and vertical ground motion, a stress drop of 120 bars is used along with a source depth of 10 km. A kappa value of 0.003 sec is used for the vertical motions, one-half the median hard rock shear-wave value of 0.006 sec (Table 3-1). Appropriate velocities and densities are taken from each crustal model at a depth of 10 km. The $Q(f)$ models used are appropriate to each crustal model (Table 3-1) and assume shear-wave propagation. This is not an important issue since motions are computed out to 50 km only (the main focus is within 20 km) and the crustal $Q(f)$ models have very little effect at these distances in the frequency range of interest, 1-34 Hz.

6.6.4.1 Outcrop V/H Ratios. Since the V/H response spectral ratios for outcrop motions are independent of strain level, they are computed for a single magnitude, M 6.5, at a suite of epicentral distances. As with the soil profiles, the crustal velocities should be randomized and median and one sigma estimates of the ratios computed. However, probabilistic analyses

have not been performed on the crustal velocities in a manner similar to that on the profiles (Appendix 6.A). As a result, only deterministic results are available for the standard crustal models and generic median profiles.

Figures 6-46 and 6-47 show the V/H ratios for the Midcontinent and Gulf Coast crustal models for distances 1, 5, 10, 15, 20, 30, 40, and 50 km. As expected, the ratios are largest at the closest distances. The ratios for 1 km and 5 km are close to one another for both crustal models and exceed 1 for frequencies above about 20 Hz. The ratios at greater distances decrease rapidly as the incidence angle increases and reach a nearly constant value around 0.6-0.8 for frequencies below 20-30 Hz. This range is in good agreement with that of Boore and Atkinson (1992) who found an average V/H value of about 0.65 over the frequency range of 1-15 Hz for a suite of ECTN recordings of Lg waves at distances exceeding about 200 km.

The rock V/H ratios depend strongly on the velocity gradient and are lower for crustal models with steep velocity gradients in the shallow crust. This has the effect of steepening the raypaths through refraction, thereby enriching the horizontal components relative to the vertical. This is demonstrated in the V/H ratios computed for the Wald et al. (1991) crustal model which is appropriate for California using the same magnitude, source depth, and set of distances. Figure 6-48 shows the ratios computed for the Wald et al. (1991) crust and, except for the large resonances, which should be averaged out by randomizing the velocities, the ratios are, on average, substantially less than the corresponding eastern ratios.

These results of linear analyses suggest that at hard rock sites, such as the Midcontinent and Gulf Coast crustal models, V/H ratios are expected to be larger than corresponding ratios at soft rock sites, providing the soft rock remains linear at high levels of input motion. There is empirical evidence, however, that soft rock sites display characteristics of nonlinear response in V/H ratios showing large V/H ratios at close distances for large magnitude earthquakes (Section 6.6.5). These large ratios are not expected at hard rock sites and illustrate the inappropriateness of applying empirical results from one tectonic regime to another.

The peak acceleration ratios are shown in Table 6-8 for both crustal structures. At the closest distance, 1 km, the ratio is about 1 and decreases rapidly out to about 20 km where it stabilizes to about 0.75-0.80.

6.6.4.2 Soil V/H Ratios. To illustrate the effects of nonlinear soil response on the computed V/H ratios for each profile category, four peak acceleration levels of horizontal outcrop motions are used: 0.10, 0.30, 0.50, and 1.00g. The magnitudes (M) and epicentral distances are listed in Table 6-3. Table 6-8 shows the peak acceleration ratios and Figures 6-49 through 6-53 show the V/H ratios for the five site categories (1-5).

For all of the categories, at the lowest level of control motion, the V/H ratios remain about 0.8 or less for frequencies less than about 30 Hz. At higher levels of motion, and for progressively deeper profiles, the ratios can be significantly greater than one at high frequencies. The large V/H ratios at high frequency for Categories 2-5 at high levels of control motion are a consequence of high levels of strain iterated damping in the horizontal motions coupled with a linear analyses using low damping (about 1%, Table 6-7) in the vertical analyses. Also, not randomizing the profiles and developing median ratios exaggerates the resonance effects of the soil column. This is shown in the sharp peaks and troughs seen in the ratios.

The high peak acceleration ratios for Categories 4 and 5 (Table 6-8) at the 1.0g control motion also reflect these effects. As a result of these factors, the large values shown in the ratios are extreme effects and do not represent expected median values. Because of this and due to the preliminary nature of the model, the concentration is on the general trends shown in the V/H ratios and recommendations will be based on these features.

6.6.5 Comparison Of Computed V/H Ratios To Empirical Ratios

Due to the lack of a sufficient number of recordings (Section 2), meaningful comparisons to empirical V/H ratios for Eastern North America sites is limited to distant sites and Lg waves. As suggested in Section 6.6.4.1 on outcrop V/H ratios, the ratios computed for both the Midcontinent and Gulf Coast crustal models at distances beyond about 20 km show general

accord with the empirical ratios of Boore and Atkinson (1992) out to the limit of their data, 15 Hz. For comparisons at closer distances and for soil sites, the only source of data are Western North America recordings (Appendix 2.C).

As with the comparisons of computed amplification factors to empirical Western North America amplification factors (Section 6.5.1), complications arise due to differences in bedrock velocities, crustal velocity gradients, and spectral content of the control motions (Section 6.3.5). However, the same general trends should exist except that the Western North America V/H ratios are expected to be generally lower than the eastern ratios due to higher velocity gradients typical in the shallow western crust.

6.6.5.1 Outcrop V/H Ratios. Figure 6-54 shows the empirical V/H ratios for the magnitude range of M 7.0-7.3 at four median distances ranging from 3.5 km to 74.7 km (Appendix 2.C). At the largest distance, and lowest level of motion, the ratio is in the range of 0.4 to about 0.7 at low frequencies, similar to that computed for the Wald et al. crust. This range reflects the expected lower V/H ratios for typical Western North America crustal structure. An unexpected feature however, is the strong distance and frequency dependence shown in the empirical rock outcrop V/H ratios. These ratios more closely resemble a smoothed version of those computed for the soil sites. Of particular interest is the increase in the ratio going from a median distance of 74.7 km to 37.8 km. This large change may be associated with a change in predominant wave type or to nonlinear rock response.

Some support does exist for nonlinear effects at rock sites. Gibbs et al. (1992) measure a significant increase in kappa values (suggesting lower velocity and higher damping) for earthquakes recorded at the rock site Gilroy #1 as the level of motion increases. Laboratory dynamic testing of soft rocks shows modulus reduction and damping strain dependencies very similar to that of soils (Ken Stokoe, personal communication). Additionally, recent drilling and logging at a number of rock sites which recorded the 1989 Loma Prieta earthquake (CUREE) show shallow velocities not unlike soil sites.

The empirical results for Western North America show a lower V/H ratio at large distance

than the eastern ratios, possibly reflecting the differences in shallow crustal velocity gradients. For the softer western rock sites, a strong dependence on distance, or alternatively level of motion, is shown in the V/H ratios and results in values greater than one at very close distances (large motions) for frequencies above about 6 Hz. This strong dependence may be due to the nonlinear response of the soft rock and is not expected at hard rock eastern sites.

6.6.5.2 Soil V/H Ratios. The Western North America empirical ratios for deep firm soils are shown in Figure 6-55 for the same magnitude and distance ranges. The deep soils show very similar trends to the rock V/H ratios with somewhat more of a distance or level of motion dependence. At low frequencies, the ratio is in the range of about 0.3-0.5, slightly lower than the rock ratios for the same frequency range. At closer distances, the ratios increase more rapidly with frequency than the rock ratios but reach about the same levels. The trend is similar to that shown in the eastern ratios, particularly for the deep Category 5 (500 ft) profile (Figure 6-53). Both the empirical and analytic all-derived ratios (Category 5) show a dip near 1 Hz, increase with increasing frequency, and then turn over. The high-frequency peak or plateau (in the empirical) occurs near 10 Hz in the empirical and near 50 Hz in the analytical. A similar feature was noted in the comparisons of empirical western and analytical eastern amplification factors (Section 6.5.1) and is related to the differences in the eastern and western kappa values for the vertical motions.

The deep soil empirical and analytical V/H ratios show very similar trends with frequency and with distance. The analytic ratios show exaggerated features because no averaging process has been done over profile compression and shear-wave velocities and damping, profile depth, depth of water table, Poisson's ratio, and modulus reduction and damping curves. Contributing to the difference at high frequency is the difference in energy content of the control motions. In view of these factors, the very simple model generally provides an accurate reflection of the trends seen in the empirical V/H ratios and may be used to suggest guidelines in estimating vertical component strong ground motions in applications to eastern rock and soil conditions.

6.6.6 Recommendations

Results of the modeling, although of a preliminary nature, indicate that V/H ratios for eastern sites are likely to be somewhat higher than V/H ratios for western United States under comparable site conditions, source sizes, and distances. The exception is at rock sites and at higher levels of motions where western empirical V/H ratios show possible effects of nonlinear response resulting in large ratios at high frequency. Because of the hard rock conditions implied in the Midcontinent and Gulf Coast crustal models, similar trends are not expected to occur at these sites with the V/H ratios remaining less than one at large distance and near to slightly above one at close distances and at high frequencies.

The modeling at soil sites shows exaggerated features in the V/H ratios which are due primarily to purely deterministic analyses using only a single set of parameters as well as maintaining low ($\approx 1\%$) compression-wave damping, even at very high levels of motion. The results do, however, suggest that large vertical component motions are possible at close distances for large magnitude sources, particularly for deeper profiles. As a result, it is not recommended that conventional approaches to specifying vertical component motions which are based largely on western empirical as well as analytical analyses be used. It is recommended rather that the following general guidelines be implemented recognizing that they are based on trends shown in the modeling results and do not represent the confidence inherent in empirical well calibrated and mature analytical results.

For rock sites in Eastern North America, the following recommendations are made:

- 1) For distances beyond 20 km, the V/H ratio should be 0.7,
- 2) For distances between 20 km and 10 km, the V/H ratio should be 0.7 up to 20 Hz and increase to 1.0 from 20 Hz to 30 Hz,
- 3) For distances within 10 km, the V/H ratio should be 1 up to 20 Hz and increase to 1.2 from 20 Hz to 30 Hz.

The recommended V/H ratios are shown diagrammatically in Figure 6-56* and are meant to apply from 1-34 Hz. For peak acceleration V/H ratios, a value of 0.8 is recommended for distances exceeding 10 km and 1.0 for closer distances. These values are based on the computed peak acceleration ratios listed in Table 6-8.

For applications to soil sites, the V/H ratios for each level of control motion were averaged over the five categories. The combination is justified on the basis of the preliminary nature of the model and simplifies the recommendations into two main categories: rock and soil. Figure 6-57 shows the V/H ratios for the combined categories at each control motion (horizontal component rock outcrop) level: 0.10, 0.30, 0.50, and 1.00g. Based on these soil V/H ratios, the following recommendations are made for the soil site V/H ratios in Eastern North America:

- 1) For control motions less than 0.10g, the V/H ratio should be 0.7,
- 2) For control motions between 0.10g, and less than 0.30g, the V/H ratio should be 0.7 up to 20 Hz and increase to 1.0 from 20 Hz to 30 Hz,
- 3) For control motions between 0.30g, and less than 0.50g, the V/H ratio should be 0.7 to 20 Hz and increase to 1.2 from 20 Hz to 30 Hz,
- 4) For control motions of 0.50g and greater, the V/H ratio should be 0.7 up to 10 Hz and increase to 1.5 from 10 Hz to 20 Hz.

The recommended V/H ratios for soil sites are shown in Figure 6-58*. As with the rock sites ratios, the frequency range of applicability is from 1 to 34 Hz.

For peak acceleration, based on Table 6-8, the following guidelines are offered. For control motions up to 0.30g, a value of 0.7 is recommended, above 0.30g and up to 0.50g the ratio should be increased to 0.8. At control motions of 0.5g and above a value of 1.2 is recommended.

It should be emphasized that these recommendations for V/H ratios are based on a single preliminary model which has not matured with applications by other individuals to a wide variety of cases. Also results of other models have not been incorporated into the

*The figures show a logarithmic transition of levels for convenient comparison to other V/H plots. In practice either a logarithmic, semi-logarithmic, or linear transition may be implemented.

recommendations. The guidelines are intended to reflect these uncertainties and still provide a reasonable basis for the specification of vertical component strong ground motions at both rock and soil sites on a generic basis in the central, eastern, and southeastern United States.

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Table 6-1

EARTHQUAKES AND STATIONS USED FOR COMPARISON
OF ENA AND WNA 5% DAMPED RESPONSE SPECTRAL SHAPES

Earthquake	Date	Magnitude (M_L)	(M)	Source Depth(km)	Epicentral Distance(km)	Station	USGS No.	Reprocessed Average Horizontal Peak Acceleration (g)
San Fernando	710209	6.4	6.6	8.4	23.0	Lake Hughes 12	128	0.316
					26.3	Lake Hughes 9	127	0.147
					27.3	Lake Hughes 4	126	0.184
					33.8	Griffith Park	141	0.185
					36.1	Seis. Lab.	266	0.151
					43.2	Santa Anita	104	0.194
Nahanni	851223	6.4 (m_b)	6.8**	18.0	8.0	Site #1		1.176
					9.0	Site #2		0.450
					20.0	Site #3		0.202
Coalinga Aftershocks								
A	83050322	3.8		7.64	8.54	LLN		0.0053
D	83050700	3.9		8.92	8.39	LLN		0.0189
K	83051013	3.9		4.79	12.46	LLN		0.0036
K3	83051405	3.9		11.18	5.56	VEW		0.0269
M	83051414	3.9		9.15	4.94	LLN		0.0084
New Brunswick	820331	4.01*		5***	4	Mitchell Road		0.2000

* After Choy and Boatwright (1988).

** After Boore and Atkinson (1987).

***After Toro and McGuire (1987).

Table 6-2

SITE CATEGORIES AND DEPTH RANGES

Category	Median Depth (ft)	Range*(ft)
1	20	10-30
2	50	31-80
3	120	81-180
4	250	181-400
5	500	401+

*Range of profile depth over which category applies as well as range of depth randomization for each category (Category 5 was randomized over the range of 401-1000 ft).

Table 6-3

MAGNITUDES (M) AND DISTANCES USED
IN OUTCROP MOTIONS

$A_p(g)$	$f_p^*(Hz)$	$V_p^{**}(cm/sec)$	$f_p^*(Hz)$	$V/A(cm/sec/g)$	M	R(km)
0.05	17.47	2.63	2.32	52.57	6.0	58.75
0.10	20.58	4.62	2.66	46.14	6.0	35.32
0.20	22.87	8.40	2.94	41.97	6.0	20.15
0.30	23.82	12.10	3.07	40.34	6.0	14.20
0.50	23.93	26.14	2.35	52.27	6.5	12.25
0.75	23.93	49.31	1.86	65.72	7.0	11.52
1.00	23.79	77.74	1.59	77.73	7.5	11.99
1.25	24.16	95.61	1.62	76.46	7.5	9.78

*Predominant frequencies associated with peak acceleration and peak particle velocity estimated by random process theory.

V_p^{**} Peak particle velocity

Outcrop motions are computed assuming a halfspace with a source region shear-wave velocity and density of 3.5 km/sec and 2.7 cgs, a stress drop of 120 bars, a kappa value of 0.006 sec, and a $Q(f) = 670 f^{0.33}$.

Table 6-4

MEDIAN AMPLIFICATION FACTORS AND STANDARD DEVIATIONS COMPUTED FOR 5% DAMPED RESPONSE SPECTRA, PEAK ACCELERATION, AND PEAK PARTICLE VELOCITY

	Category 1		Category 2		Category 3		Category 4		Category 5		Input ¹
Frequencies (Hz)	20' (10-30')		50' (31-80')		120' (81-180')		250' (181-400')		500' (401- + ')		(rock outcrop) (g)
	Median Sigma ²		Median Sigma		Median Sigma		Median Sigma		Median Sigma		
1.00	1.02	0.02	1.05	0.03	1.18	0.14	1.67	0.36	2.10	0.36	0.05
	1.02	0.02	1.05	0.04	1.19	0.16	1.71	0.37	2.10	0.37	0.10
	1.02	0.02	1.06	0.05	1.23	0.21	1.77	0.38	2.08	0.38	0.20
	1.03	0.03	1.07	0.06	1.26	0.24	1.82	0.38	2.07	0.39	0.30
	1.03	0.05	1.09	0.09	1.33	0.29	1.91	0.39	2.00	0.42	0.50
	1.05	0.09	1.13	0.14	1.39	0.27	2.01	0.39	1.81	0.50	0.75
	1.09	0.16	1.19	0.21	1.45	0.33	2.01	0.37	1.81	0.46	1.00
	1.10	0.18	1.23	0.23	1.52	0.35	2.05	0.38	1.72	0.46	1.25
1.50	1.03	0.03	1.10	0.07	1.44	0.35	2.25	0.36	1.82	0.40	0.05
	1.04	0.04	1.11	0.08	1.48	0.37	2.27	0.36	1.83	0.40	0.10
	1.04	0.05	1.13	0.11	1.53	0.35	2.30	0.36	1.80	0.39	0.20
	1.05	0.06	1.16	0.14	1.58	0.35	2.32	0.35	1.78	0.38	0.30
	1.08	0.10	1.22	0.21	1.70	0.38	2.29	0.36	1.72	0.36	0.50
	1.12	0.19	1.31	0.30	1.79	0.39	2.15	0.36	1.62	0.46	0.75
	1.16	0.20	1.35	0.29	1.76	0.41	1.95	0.38	1.56	0.39	1.00
	1.17	0.19	1.39	0.32	1.77	0.41	1.86	0.39	1.45	0.43	1.25

Table 6-4 (cont.)

MEDIAN AMPLIFICATION FACTORS AND STANDARD DEVIATIONS COMPUTED FOR 5% DAMPED RESPONSE SPECTRA, PEAK ACCELERATION, AND PEAK PARTICLE VELOCITY

	Category 1		Category 2		Category 3		Category 4		Category 5		Input ¹
Frequencies (Hz)	20' (10-30')		50' (31-80')		120' (81-180')		250' (181-400')		500' (401- + ')		(rock outcrop) (g)
	Median Sigma ²		Median Sigma		Median Sigma		Median Sigma		Median Sigma		
2.50	1.08	0.08	1.30	0.21	2.15	0.39	2.12	0.32	1.98	0.30	0.05
	1.09	0.10	1.34	0.26	2.19	0.39	2.10	0.31	1.96	0.30	0.10
	1.12	0.13	1.42	0.33	2.21	0.37	2.05	0.30	1.91	0.30	0.20
	1.15	0.19	1.48	0.36	2.23	0.35	1.99	0.32	1.85	0.29	0.30
	1.22	0.29	1.58	0.37	2.25	0.38	1.83	0.36	1.73	0.35	0.50
	1.24	0.24	1.62	0.40	2.11	0.40	1.66	0.43	1.52	0.48	0.75
	1.25	0.33	1.58	0.44	1.81	0.47	1.51	0.48	1.37	0.47	1.00
	1.28	0.36	1.51	0.44	1.65	0.49	1.47	0.46	1.23	0.51	1.25
3.45	1.16	0.17	1.66	0.40	2.50	0.34	1.94	0.34	2.03	0.32	0.05
	1.19	0.21	1.73	0.41	2.47	0.33	1.92	0.35	2.01	0.31	0.10
	1.25	0.28	1.81	0.42	2.43	0.35	1.85	0.36	1.97	0.33	0.20
	1.31	0.35	1.87	0.44	2.38	0.36	1.78	0.35	1.91	0.34	0.30
	1.34	0.30	1.91	0.47	2.19	0.41	1.62	0.34	1.66	0.40	0.50
	1.38	0.39	1.78	0.46	1.83	0.48	1.50	0.39	1.33	0.54	0.75
	1.32	0.42	1.61	0.47	1.54	0.57	1.34	0.43	1.10	0.47	1.00
	1.31	0.39	1.51	0.48	1.43	0.57	1.27	0.44	0.94	0.51	1.25

Table 6-4 (cont.)

MEDIAN AMPLIFICATION FACTORS AND STANDARD DEVIATIONS COMPUTED FOR 5% DAMPED RESPONSE SPECTRA, PEAK ACCELERATION, AND PEAK PARTICLE VELOCITY

	Category 1		Category 2		Category 3		Category 4		Category 5		Input ¹
Frequencies (Hz)	20' (10-30')		50' (31-80')		120' (81-180')		250' (181-400')		500' (401- +)		(rock outcrop) (g)
	Median Sigma ²		Median Sigma		Median Sigma		Median Sigma		Median Sigma		
5.00	1.41	0.43	2.34	0.49	2.22	0.35	1.95	0.31	1.91	0.30	0.05
	1.46	0.43	2.33	0.46	2.18	0.38	1.91	0.31	1.84	0.30	0.10
	1.51	0.41	2.28	0.44	2.09	0.41	1.82	0.31	1.73	0.32	0.20
	1.54	0.41	2.23	0.47	1.99	0.40	1.73	0.30	1.65	0.34	0.30
	1.58	0.45	1.98	0.42	1.77	0.40	1.56	0.37	1.38	0.40	0.50
	1.51	0.45	1.73	0.41	1.48	0.45	1.33	0.45	1.03	0.48	0.75
	1.37	0.50	1.53	0.47	1.25	0.52	1.14	0.51	0.84	0.46	1.00
	1.35	0.51	1.41	0.53	1.11	0.54	1.04	0.49	0.71	0.50	1.25
7.50	1.87	0.51	2.35	0.41	2.09	0.34	2.03	0.35	1.86	0.28	0.05
	1.89	0.50	2.26	0.39	2.08	0.35	1.98	0.35	1.79	0.29	0.10
	1.93	0.50	2.12	0.35	1.96	0.34	1.85	0.33	1.63	0.30	0.20
	1.90	0.51	2.03	0.35	1.82	0.31	1.74	0.32	1.47	0.31	0.30
	1.79	0.49	1.83	0.41	1.47	0.33	1.43	0.38	1.10	0.37	0.50
	1.61	0.55	1.58	0.53	1.22	0.44	1.12	0.45	0.78	0.55	0.75
	1.44	0.69	1.42	0.64	1.00	0.58	0.90	0.52	0.62	0.52	1.00
	1.38	0.69	1.29	0.69	0.89	0.65	0.80	0.53	0.51*	0.54	1.25

Table 6-4 (cont.)

MEDIAN AMPLIFICATION FACTORS AND STANDARD DEVIATIONS COMPUTED FOR 5% DAMPED RESPONSE SPECTRA, PEAK ACCELERATION, AND PEAK PARTICLE VELOCITY

	Category 1		Category 2		Category 3		Category 4		Category 5		Input ¹
Frequencies (Hz)	20' (10-30')		50' (31-80')		120' (81-180')		250' (181-400')		500' (401- +)		(rock outcrop) (g)
	Median Sigma ²		Median Sigma		Median Sigma		Median Sigma		Median Sigma		
10.00	2.28	0.54	2.15	0.38	2.15	0.33	2.01	0.26	1.85	0.26	0.05
	2.26	0.54	2.10	0.36	2.01	0.31	1.91	0.27	1.72	0.27	0.10
	2.16	0.50	2.04	0.37	1.81	0.33	1.75	0.30	1.49	0.30	0.20
	2.05	0.49	1.91	0.39	1.66	0.34	1.60	0.33	1.31	0.34	0.30
	1.85	0.50	1.66	0.45	1.37	0.41	1.25	0.40	0.95	0.44	0.50
	1.60	0.61	1.48	0.53	1.10	0.56	0.97	0.48	0.65	0.58	0.75
	1.36	0.70	1.21	0.64	0.89	0.69	0.75	0.54	0.50*	0.51	1.00
	1.27	0.70	1.06	0.71	0.77	0.68	0.66	0.54	0.41*	0.52	1.25
15.00	2.40	0.41	2.04	0.29	2.07	0.33	1.91	0.24	1.66	0.27	0.05
	2.34	0.42	1.90	0.29	1.93	0.34	1.76	0.24	1.50	0.29	0.10
	2.20	0.44	1.75	0.32	1.70	0.37	1.51	0.28	1.25	0.34	0.20
	2.03	0.46	1.67	0.34	1.53	0.37	1.32	0.34	1.07	0.39	0.30
	1.79	0.52	1.42	0.41	1.22	0.47	1.02	0.46	0.72	0.48	0.50
	1.51	0.62	1.18	0.53	0.92	0.60	0.76	0.55	0.47*	0.55	0.75
	1.28	0.78	1.01	0.64	0.72	0.70	0.58*	0.59	0.37*	0.46	1.00
	1.16	0.82	0.88	0.71	0.61	0.69	0.50*	0.59	0.30*	0.45	1.25

Table 6-4 (cont.)

MEDIAN AMPLIFICATION FACTORS AND STANDARD DEVIATIONS COMPUTED FOR 5% DAMPED RESPONSE SPECTRA, PEAK ACCELERATION, AND PEAK PARTICLE VELOCITY

	Category 1		Category 2		Category 3		Category 4		Category 5		Input ¹
Frequencies (Hz)	20' (10-30')		50' (31-80')		120' (81-180')		250' (181-400')		500' (401- + ')		(rock outcrop) (g)
	Median Sigma ²		Median Sigma		Median Sigma		Median Sigma		Median Sigma		
25.00	2.31	0.36	2.05	0.27	1.90	0.24	1.74	0.25	1.39	0.26	0.05
	2.21	0.38	1.93	0.30	1.76	0.27	1.55	0.29	1.20	0.29	0.10
	1.97	0.36	1.78	0.32	1.48	0.30	1.27	0.38	0.94	0.37	0.20
	1.81	0.38	1.63	0.36	1.26	0.36	1.07	0.43	0.76	0.40	0.30
	1.56	0.43	1.32	0.46	0.93	0.47	0.76	0.49	0.50*	0.43	0.50
	1.29	0.57	1.03	0.58	0.68	0.56	0.56*	0.53	0.35*	0.46	0.75
	1.07	0.71	0.84	0.67	0.53*	0.64	0.44*	0.54	0.29*	0.38	1.00
	0.97	0.76	0.72	0.72	0.46*	0.64	0.38*	0.52	0.24*	0.37	1.25
34.00	2.12	0.27	1.96	0.21	1.78	0.19	1.58	0.18	1.29	0.23	0.05
	2.00	0.28	1.81	0.23	1.57	0.23	1.38	0.23	1.07	0.27	0.10
	1.84	0.30	1.60	0.28	1.31	0.32	1.09	0.33	0.79	0.33	0.20
	1.70	0.33	1.45	0.33	1.12	0.38	0.90	0.37	0.64	0.35	0.30
	1.45	0.44	1.17	0.45	0.82	0.48	0.65	0.44	0.44*	0.36	0.50
	1.21	0.60	0.93	0.58	0.61	0.55	0.49*	0.46	0.32*	0.41	0.75
	1.00	0.75	0.76	0.67	0.49*	0.61	0.39*	0.46	0.27*	0.34	1.00
	0.89	0.79	0.65	0.71	0.41*	0.59	0.34*	0.45	0.24*	0.35	1.25

Table 6-4 (cont.)

**MEDIAN AMPLIFICATION FACTORS AND STANDARD DEVIATIONS COMPUTED FOR 5% DAMPED RESPONSE
SPECTRA, PEAK ACCELERATION, AND PEAK PARTICLE VELOCITY**

	Category 1		Category 2		Category 3		Category 4		Category 5		Input ¹
Frequencies (Hz)	20' (10-30')		50' (31-80')		120' (81-180')		250' (181-400')		500' (401- + ')		(rock outcrop) (g)
	Median Sigma ²		Median Sigma		Median Sigma		Median Sigma		Median Sigma		
Ap (site)/ Ap (rock)	2.29	0.17	2.15	0.17	2.12	0.19	1.95	0.17	1.82	0.20	0.05
	2.28	0.15	2.08	0.16	1.98	0.19	1.79	0.17	1.63	0.20	0.10
	2.18	0.15	1.93	0.17	1.76	0.21	1.56	0.19	1.38	0.21	0.20
	2.05	0.18	1.80	0.19	1.59	0.23	1.40	0.21	1.21	0.23	0.30
	1.83	0.25	1.58	0.25	1.34	0.28	1.17	0.26	0.99	0.25	0.50
	1.60	0.35	1.37	0.33	1.13	0.34	0.99	0.30	0.82	0.26	0.75
	1.41	0.45	1.21	0.39	0.97	0.40	0.86	0.33	0.73	0.26	1.00
	1.32	0.47	1.10	0.43	0.88	0.40	0.78	0.33	0.62	0.47	*1.25
PGV(site)/ PGV(rock)	1.23	0.10	1.33	0.12	1.53	0.17	1.71	0.18	1.88	0.19	0.05
	1.25	0.10	1.35	0.12	1.55	0.16	1.71	0.17	1.87	0.19	0.10
	1.27	0.10	1.36	0.12	1.55	0.15	1.70	0.16	1.83	0.18	0.20
	1.27	0.09	1.36	0.11	1.54	0.14	1.68	0.15	1.80	0.17	0.30
	1.18	0.05	1.24	0.08	1.37	0.11	1.48	0.14	1.62	0.16	0.50
	1.13	0.03	1.16	0.05	1.24	0.08	1.34	0.12	1.46	0.14	0.75
	1.09	0.03	1.12	0.04	1.18	0.07	1.27	0.12	1.38	0.13	1.00
	1.09	0.03	1.11	0.04	1.17	0.07	1.25	0.09	1.37	0.24	1.25

¹For levels of control motion between those shown, linear interpolation may be used to determine the appropriate amplification factors.

²Sigma (ln)

*A lower limit of 0.6 should be substituted for these values.

Table 6-5

REPORTED ANALYTICAL AMPLIFICATION FACTORS

Analytical Results								
Frequency (Hz)	2 31'-80'	L ³ 25'-80'	3 81'-180'	L ³ 81'-180'	4 180'-400'	L ³ 181'-300'	5 400'+	L ³ 650'
1.0	1.13	1.10	1.39	1.30	2.01	1.50	1.81	1.70
2.5	1.62	1.20	2.11	1.70	1.66	1.90	1.55	1.50
5.0	1.73	1.65	1.48	1.85	1.33	1.50	1.03	1.10
10.0	1.48	1.90	1.10	1.70	0.97	1.30	0.65	0.80
25.0	1.03	1.70	0.68	1.70	0.60	1.40	0.60	0.90
A _p site/ A _p rock	1.37	1.70	1.13	1.70	0.99	1.40	0.82	1.00
V _p site/ V _p rock	1.16	-----	1.24	-----	1.34	-----	1.46	-----

2, 3, 4, 5: values taken from Table 6-4 for input motion of 1.25 g (corresponds to average damping of approximately 7%)

L³ Lawrence Livermore National Laboratory with 7% median damping (Bernreuter et al., 1985)

Table 6-6
LOMA PRIETA MODELING SUMMARY FOR VERTICAL MOTIONS

Name	Label	$\kappa(\text{sec})^{**}$	R(km)
Corralitos	COR	0.028	12.0
Los Gatos	LGPC	0.012	12.4
Branciforte Dr.	BRAN	0.020	15.6
Gilroy 1	GL1	0.012	19.2
Gilroy 2	GL2*	0.029	20.0
Santa Cruz	UCS	0.020	20.0
UCSC-Seismic Lab	UCSC	0.020	23.3
Gilroy 6	GL6	0.025	26.8
SLAC	SLA	0.030	30.5
Redwood City-Canāda	RDC	0.060	39.9
SAGO South	SAS	0.050	40.8
APEEL 7	A07	0.050	45.6
APEEL 10	A10	0.050	45.6
Monterrey City Hall	MON	0.030	45.6
Belmont-2 story	BEL	0.040	47.5
APEEL 2E, CUSH	A2E	0.020	54.3
Sierra Point	SSP	0.020	66.1
SF Diamond Heights	SFD	0.020	74.0
Piedmont J.H.	PHS	0.025	75.0
SF Rincon	SFR	0.020	76.9
Yerba Buena Island	YBI	0.040	77.9
SF Pacific Heights	SFH	0.040	78.9
SF Telegraph Hill	SFT	0.020	78.9
Treasure Island	TRI*	0.029	79.9
SF Cliff House	SFC	0.040	80.9

*soil sites

**kappa values are relative to the fixed $Q(f)$ of $150 f^{0.6}$. For soil sites the average kappa value of 0.029 sec is used for the shallow crust beneath the profiles.

Table 6-7

SHEAR- AND LINEAR COMPRESSIONAL-WAVE DAMPING VERSES DEPTH
USED IN VERTICAL SITE RESPONSE ANALYSES

Depth Range (ft)	Shear-Wave Damping Ratio (%),Q	Compressional-Wave Damping Ratio (%),Q
0-120	1.0, 50.0	1.0, 50.0
121-250	0.8, 62.5	0.8, 62.5
251-500	0.7, 71.4	0.7, 71.4

Table 6-8

VERTICAL-TO-HORIZONTAL PEAK ACCELERATION RATIOS

Site	Distance (km)	Vpga/Hpga
Midcontinent	1	1.03
	5	0.96
	10	0.86
	15	0.80
	20	0.78
	30	0.77
	40	0.76
	50	0.75
Gulf-Coast	1	0.99
	5	0.94
	10	0.84
	15	0.82
	20	0.81
	30	0.79
	40	0.77
	50	0.76
Soil Site	Rock Outcrop (g)	Vpga/Hpga
1	0.10	0.48
	0.30	0.53
	0.50	0.57
	1.00	0.65
2	0.10	0.54
	0.30	0.85
	0.50	0.67
	1.00	0.83
3	0.10	0.58
	0.30	0.46
	0.50	0.77
	1.00	1.01
4	0.10	0.62
	0.30	0.76
	0.50	0.87
	1.00	1.17
5	0.10	0.66
	0.30	0.64
	0.50	0.96
	1.00	1.33

Figure 6-1. Average 5% damping response spectral shapes (S_a/a) computed from motions recorded on different soil conditions (after Seed, et al., 1976).

Figure 6-2. Average 5% damping response spectral shapes (S_a/a_{max}) computed from motions recorded on rock sites at close distances to $M = 6.4$ earthquakes (top figure) and $M = 4.0$ earthquakes (bottom figure). In each figure the solid line corresponds to motions recorded in WNA, dashed line to motions recorded in ENA. (See Table 6-1 for a list of earthquakes, sites, distances, and average peak accelerations).

Figure 6-3. Standard soil profile appropriate for the Central and Eastern United States sites (gradient). Site categories 1-5 are indicated by their respective soil column depths. Constant shear-wave velocity profiles represent averages (over travel time) of the gradient profiles for each site category.

Figure 6-4. Randomized shear-wave velocity profiles (velocity and depth randomized) for Category 1 with a median depth of 20 ft and a depth range of 10-30 ft. Median soil profile is dashed and median bedrock velocity is 6000 ft/sec.

Figure 6-5. Randomized shear-wave velocity profiles (velocity and depth randomized) for Category 2 with a median depth of 50 ft and a depth range of 31-80 ft. Median soil profile is dashed and median bedrock velocity is 6000 ft/sec.

Figure 6-6. Randomized shear-wave velocity profiles (velocity and depth randomized) for Category 3 with a median depth of 120 ft and a depth range of 81-180 ft. Median soil profile is dashed and median bedrock velocity is 6000 ft/sec.

Figure 6-7. Randomized shear-wave velocity profiles (velocity and depth randomized) for Category 4 with a median depth of 250 ft and a depth range of 181-400 ft. Median soil profile is dashed and median bedrock velocity is 6000 ft/sec.

Figure 6-8. Randomized shear-wave velocity profiles (velocity and depth randomized) for Category 5 with a median depth of 500 ft and a depth range of 401 + ft (lower limit on depth randomization is 1000 ft). Median profile is dashed and median bedrock velocity is 6000 ft/sec.

Figure 6-9. Modulus reduction and damping curves (median) used in the site response analyses. Damping curves are the same for depths ranging from 0 to 120 ft.

Figure 6-10. Example of randomized modulus reduction and damping curves (20) taken from the suite of 50 used in the analyses. Dashed lines are upper- and lower-bounds. Depth range is 0-20 ft.

Figure 6-11. Example of randomized modulus reduction and damping curves (20) taken from the suite of 50 used in the analyses. Dashed lines are upper- and lower-bounds. Depth range is 21-50 ft.

Figure 6-12. Example of randomized modulus reduction and damping curves (20) taken from the suite of 50 used in the analyses. Dashed lines are upper- and lower-bounds. Depth range is 51-120 ft.

Figure 6-13. Example of randomized modulus reduction and damping curves (20) taken from the suite of 50 used in the analyses. Dashed lines are upper- and lower-bounds. Depth range is 121-250 ft.

Figure 6-14. Example of randomized modulus reduction and damping curves (20) taken from the suite of 50 used in the analyses. Dashed lines are upper- and lower-bounds. Depth range is 251-500 ft.

Figure 6-15. Example of randomized modulus reduction and damping curves (20) taken from the suite of 50 used in the analyses. Dashed lines are upper- and lower-bounds. Depth range is 501-1000 ft.

Figure 6-16. Pseudo-absolute acceleration response spectra (5% damping) computed for the outcrop or control motions (Table 6-3).

CATEGORY 3 (120 FT)

- LEGEND
- CONTROL MOTION 0.10 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
 - CONTROL MOTION 0.10 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
 - CONTROL MOTION 0.10 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure 6-17. Category 3 (81-180 ft) median and 1 sigma amplification factors for 5% damped response spectra. Control motion is 0.10 g.

CATEGORY 3 (120 FT)

- LEGEND
- CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
 - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
 - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure 6-18. Category 3 (81-180 ft) median and 1 sigma amplification factors for 5% damped response spectra. Control motion is 0.50 g.

CATEGORY 3 (120 FT)

LEGEND	
—	CONTROL MOTION 1.00 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—	CONTROL MOTION 1.00 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.
—	CONTROL MOTION 1.00 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 81 & 180 FT.

Figure 6-19. Category 3 (81-180 ft) median and 1 sigma amplification factors for 5% damped response spectra. Control motion is 1.00 g.

Figure 6-20. Plot of median 5% damped response spectral amplification factors computed for the five site categories (Table 6-2) at a frequency of 1 Hz. Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

Figure 6-21. Plot of median 5% damped response spectral amplification factors computed for the five site categories (Table 6-2) at a frequency of 2.5 Hz. Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

Figure 6-22. Plot of median 5% damped response spectral amplification factors computed for the five site categories (Table 6-2) at a frequency of 5.0 Hz. Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

Figure 6-23. Plot of median 5% damped response spectral amplification factors computed for the five site categories (Table 6-2) at a frequency of 10.0 Hz. Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

Figure 6-24. Plot of median 5% damped response spectral amplification factors computed for the five site categories (Table 6-2) at a frequency of 25.0 Hz. Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

Figure 6-25. Plot of median 5% damped response spectral amplification factors computed for the five site categories (Table 6-2) at a frequency of 34.0 Hz. Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

Figure 6-26. Plot of computed median amplification of peak acceleration for the five site categories (Table 6-2). Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

Figure 6-27. Plot of computed median amplification of peak particle velocity for the five site categories (Table 6-2). Size of symbol indicates level of input (rock outcrop) acceleration (0.1, 0.3, 0.5, 0.75, 1 g).

CATEGORY 2 (50 FT)

LEGEND

- CONTROL MOTION 0.30 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 6.0
- CONTROL MOTION 0.30 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 6.0
- CONTROL MOTION 0.30 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 6.0
- CONTROL MOTION 0.30 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 7.5
- CONTROL MOTION 0.30 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 7.5
- CONTROL MOTION 0.30 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 7.5

Figure 6-28. Effects of control motion magnitude on amplification factors. Category 2 (31-80 ft) median amplification factors computed for control motion of 0.30 g. Solid lines are for M 6.0 and dashed lines are for M 7.5.

CATEGORY 2 (50 FT)

LEGEND	
—	CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 6.5
—	CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 6.5
—	CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 6.5
- - - - -	CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 7.5
- - - - -	CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 7.5
- - - - -	CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES DEPTH RANDOMIZED BETWEEN 31 & 80 FT, M 7.5

Figure 6-29. Effects of control motion magnitude on amplification factors. Category 2 (31-80 ft) median amplification factors computed for control motion of 0.50 g. Solid lines are for M 6.5 and dashed lines are for M 7.5.

CATEGORY 5 (500 FT)

- LEGEND
- CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT

Figure 6-30. Effects of bedrock velocity on amplification factors. Category 5 (401+ft) median amplification factors computed for control motion of 0.50 g. Solid lines are with median bedrock shear-wave velocity of 6000 ft/sec and dashed lines are with 9000 ft/sec.

CATEGORY 5 (500 FT)

- LEGEND
- CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - - - CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - - - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT
 - - - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 401 & 1000 FT

Figure 6-31. Effects of bedrock velocity on amplification factors. Category 5 (401+ft) median amplification factors computed for control motion of 0.50 g. Solid lines are with median bedrock shear-wave velocity of 6000 ft/sec and dashed lines are with 3000 ft/sec.

CATEGORY 3 (120 FT)

- LEGEND
- CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT
 - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT
 - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT
 - - - CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED UNIFORM PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT
 - - - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED UNIFORM PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT
 - - - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED UNIFORM PROFILES
DEPTH RANDOMIZED BETWEEN 81 & 180 FT

Figure 6-32. Effects of uniform profiles on amplification factors. Category 3 (81-180 ft) median amplification factors computed for a control motion of 0.50 g. Solid lines are using gradient profile, dashed lines are using uniform profile.

CATEGORY 4 (250 FT)

- LEGEND
- CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 181 & 400 FT
 - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 181 & 400 FT
 - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED PROFILES
DEPTH RANDOMIZED BETWEEN 181 & 400 FT
 - CONTROL MOTION 0.50 G; + 1 SIGMA OF 50 RANDOMIZED UNIFORM PROFILES
DEPTH RANDOMIZED BETWEEN 181 & 400 FT
 - CONTROL MOTION 0.50 G; MEDIAN OF 50 RANDOMIZED UNIFORM PROFILES
DEPTH RANDOMIZED BETWEEN 181 & 400 FT
 - CONTROL MOTION 0.50 G; - 1 SIGMA OF 50 RANDOMIZED UNIFORM PROFILES
DEPTH RANDOMIZED BETWEEN 181 & 400 FT

Figure 6-33. Effects of uniform profiles on amplification factors. Category 4 (181-400 ft) median amplification factors computed for a control motion of 0.50 g. Solid lines are using gradient profile, dashed lines are using uniform profile.

CATEGORY 5 (500 FT)

LEGEND
 — WNA EMPIRICAL, ROCK = 0.06 G
 - - - ENA COMPUTED, ROCK = 0.05 G
 MEDIAN BEDROCK VELOCITY = 6000 FT/SEC
 - . - ENA COMPUTED, ROCK = 0.05 G
 MEDIAN BEDROCK VELOCITY = 3000 FT/SEC

Figure 6-34. Comparison of empirical western North America amplification factors for deep stiff soil (solid line) to computed factors for eastern North America for Category 5 (401+ft). Dashed line is with median bedrock velocity of 6000 ft/sec, dash-dotted line is with median bedrock velocity of 3000 ft/sec. Control or corresponding rock outcrop motion is 0.06 g for the empirical factors and 0.05 g for the analytical factors.

CATEGORY 5 (500 FT)

- LEGEND
- WNA EMPIRICAL, ROCK = 0.12 G
 - - - ENA COMPUTED, ROCK = 0.10 G
MEDIAN BEDROCK VELOCITY = 6000 FT/SEC
 - . - ENA COMPUTED, ROCK = 0.10 G
MEDIAN BEDROCK VELOCITY = 3000 FT/SEC

Figure 6-35. Comparison of empirical western North America amplification factors for deep stiff soil (solid line) to computed factors for eastern North America for Category 5 (401+ ft). Dashed line is with median bedrock velocity of 6000 ft/sec, dash-dotted line is with median bedrock velocity of 3000 ft/sec. Control or corresponding rock outcrop motion is 0.12 g for the empirical factors and 0.10 g for the analytical factors.

CATEGORY 5 (500 FT)

- LEGEND
- WNA EMPIRICAL, ROCK = 0.26 G
 - - - ENA COMPUTED, ROCK = 0.30 G
MEDIAN BEDROCK VELOCITY = 6000 FT/SEC
 - . - ENA COMPUTED, ROCK = 0.30 G
MEDIAN BEDROCK VELOCITY = 3000 FT/SEC

Figure 6-36. Comparison of empirical western North America amplification factors for deep stiff soil (solid line) to computed factors for eastern North America for Category 5 (401+ft). Dashed line is with median bedrock velocity of 6000 ft/sec, dash-dotted line is with median bedrock velocity of 3000 ft/sec. Control or corresponding rock outcrop motion is 0.26 g for the empirical factors and 0.30 g for the analytical factors.

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CATEGORY 5 (500 FT)

LEGEND	
————	WNA EMPIRICAL, ROCK = 0.42 G
-----	ENA COMPUTED, ROCK = 0.50 G MEDIAN BEDROCK VELOCITY = 6000 FT/SEC
- . - .	ENA COMPUTED, ROCK = 0.50 G MEDIAN BEDROCK VELOCITY = 3000 FT/SEC

Figure 6-37. Comparison of empirical western North America amplification factors for deep stiff soil (solid line) to computed factors for eastern North America for Category 5 (401+ft). Dashed line is with median bedrock velocity of 6000 ft/sec, dash-dotted line is with median bedrock velocity of 3000 ft/sec. Control or corresponding rock outcrop motion is 0.42 g for the empirical factors and 0.50 g for the analytical factors.

CATEGORY 5 (500 FT)

- LEGEND
- WNA EMPIRICAL, ROCK = 0.75 G
 - - - ENA COMPUTED, ROCK = 0.75 G
MEDIAN BEDROCK VELOCITY = 6000 FT/SEC
 - . - ENA COMPUTED, ROCK = 0.75 G
MEDIAN BEDROCK VELOCITY = 3000 FT/SEC

Figure 6-38. Comparison of empirical western North America amplification factors for deep stiff soil (solid line) to computed factors for eastern North America for Category 5 (401+ft). Dashed line is with median bedrock velocity of 6000 ft/sec, dash-dotted line is with median bedrock velocity of 3000 ft/sec. Control or corresponding rock outcrop motion is 0.75 g for the empirical factors and 0.75 g for the analytical factors.

Figure 6-39. Comparison of Boore and Joyner (1992) linear amplification factors of Fourier amplitude spectra for eastern North America S2 (stiff soil > 200 ft depth) sites (dashed line) to median response spectra factors for Categories 1-5 (solid lines). The median factors for the Categories form a family of curves beginning at low frequency for Category 5. Control motion is 0.10 g for the Categories. The Fourier amplitude factors are linear and independent of control motion amplitude.

Figure 6-40. Comparison of Boore and Joyner (1992) linear amplification factors of Fourier amplitude spectra for eastern North America S2 (stiff soil > 200 ft depth) sites (dashed line) to median response spectra factors for Categories 1-5 (solid lines). The median factors for the Categories form a family of curves beginning at low frequency for Category 5. Control motion is 0.50 g for the Categories. The Fourier amplitude factors are linear and independent of control motion amplitude.

Figure 6-41. Plot of Poisson's ratio verses depth based on suspension log compressional- and shear-wave velocities at the Savannah River Site.

Figure 6-42. Comparison of 5% damped vertical component response spectra for the 1989 Loma Prieta earthquake. Solid lines are from recordings and dash-dot lines are model predictions, page 1 of 2.

Figure 6-42. Continued, page 2 of 2.

MODELING UNCERTAINTY FOR LOMA PRIETA VERTICALS
24 SITES

Figure 6-43. Standard error (natural log) of modeling uncertainty computed for the Loma Prieta earthquake using vertical motions. Solid line, total uncertainty; dashed line, corrected for model bias.

MODELING BIAS FOR LOMA PRIETA VERTICALS, 24 SITES

Figure 6-44. Modeling bias computed for the Loma Prieta earthquake using vertical motions. Dashed lines represent 90% confidence limits.

CRUSTAL PROFILES FOR EASTERN NORTH AMERICA

Figure 6-45. Compressional- and shear-wave velocities for the Midcontinent and Gulf-Coast crustal models.

MID-CONTINENT CRUST, M 6.5, SD=120 BARS, H=10 KM

Figure 6-46. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Midcontinent crustal model.

GULF COAST CRUST, M 6.5, SD=120 BARS, H=10 KM

Figure 6-47. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Gulf-Coast crustal model.

WALD CRUST, M 6.5, SD=100 BARS, H=10 KM

Figure 6-48. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Wald et al. (1991) crustal model.

Figure 6-49. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Category 1 (20 ft) soil profile.

Figure 6-50. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Category 2 (50 ft) soil profile.

Figure 6-51. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Category 3 (120 ft) soil profile.

Figure 6-52. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Category 4 (250 ft) soil profile.

Figure 6-53. Vertical-to-horizontal (V/H) 5% damped response spectral ratios computed for the Category 5 (500 ft) soil profile.

Figure 6-54. Empirical V/H ratios at rock sites for western North America earthquakes of magnitudes M 7.0-7.3. Peak acceleration values are for rock outcrop horizontal motions.

WNA EMPIRICAL M 7.0 - M 7.3
DEEP, FIRM SITES

Figure 6-55. Empirical V/H ratios at deep soil sites for western North America earthquakes of magnitudes M 7.0-7.3. Peak acceleration values are for rock outcrop horizontal motions.

RECOMMENDED V/H RATIOS FOR ROCK

Figure 6-56. Recommended 5% damped V/H ratios for rock sites. Frequency range of application is from 1 Hz to 34 Hz.

VERTICAL RATIOS AVERAGED OVER THE SOIL CATAGORIES
FOR EACH CONTROL MOTION LEVEL

Figure 6-57. Log average V/H ratios for all five soil Categories (20 ft-500 ft).

RECOMMENDED V/H RATIOS FOR SOIL

Figure 6-58. Recommended 5% damped V/H ratios for soil sites. Frequency range of application is from 1 Hz to 34 Hz.

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