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Program on Technology Innovation: Assessment of a Performance-Based Approach for Determining Seismic Ground Motions for New Plant Sites, V2

Volume 2: Seismic Hazard Results at 28 Sites

Technical Report

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PRODUCT DESCRIPTION

Interest in recent years in early site permits (ESPs) for new nuclear plants has prompted a reevaluation of seismic design criteria and a reexamination of the basis for current criteria. Currently, Regulatory Guide 1.165 bases seismic design requirements on probabilistic seismic hazard analyses (PSHAs) at 29 nuclear plant sites using results that were published in 1989 and 1994. Much new work has been undertaken since to better understand earthquakes in the Central and Eastern United States (CEUS) and associated strong ground motions. This study recalculates seismic hazard at 28 of the original 29 nuclear plant sites, accounting for new information as a basis for further work to redefine seismic criteria for new nuclear plants in the CEUS.

Results & Findings

This work calculates probabilistic seismic hazard at 28 nuclear plant sites in the CEUS for ground motions between the peak ground acceleration (PGA, at 100 Hz) and 1 Hz. New information on seismic sources in the CEUS has been incorporated in the probabilistic estimates, and a new comprehensive model of ground motion (quantifying both aleatory and epistemic uncertainty) has been used. The seismic hazard results define means and fractiles of spectral accelerations with annual frequencies of exceedance between 10^{-3} and 10^{-7}.

Challenges & Objective(s)

This report will be useful in establishing the basis for seismic design of new nuclear plants in the CEUS. Current regulations (Regulatory Guide 1.165) lead to overly conservative requirements for seismic design, and the current study will allow further analyses to show that performance-based methods for establishing seismic criteria (such as that proposed by a committee of the American Society of Civil Engineers) are reasonable and result in seismically safe plants.

Applications, Values & Use

These results can be used in several ways. First, the seismic design values recommended by performance-based procedures can be calculated for the 28 nuclear plants and can be compared to current design levels to evaluate consistency with current practice. Second, simple models of nuclear plant seismic behavior can be used with the seismic hazard calculated here to compare to calculated annual frequencies of seismically induced core melt from detailed probabilistic risk assessments done for existing nuclear plants. This comparison also will allow evaluations of consistency between seismic designs determined with performance-based procedures and estimates of current nuclear plant safety.

EPRI Perspective

This study could only be undertaken by an industry group such as EPRI that has a broad perspective on nuclear plant policy and that can make a substantial independent contribution to solving design issues. The current seismic design requirements of Regulatory Guide 1.165 are based on seismic hazard estimates that are approaching 20 years, and applying for exemptions on a site-by-site basis would likely be time-consuming and perhaps unsuccessful.

Approach

The approach taken here was to use new information on earthquake sources in the CEUS and on earthquake ground motion estimation and to modify earlier work published by EPRI in 1989. The three 2003 ESP applications for nuclear plants in the CEUS contain substantial, detailed, new information on seismic sources, and a large study published by EPRI in 2004 contains a comprehensive model for estimating seismic ground motions. This information leads to a comprehensive, justifiable set of assumptions for calculating probabilistic seismic hazard. For some sites, a study of dynamic site response was undertaken. Information on site properties for these sites was taken from existing final safety analysis reports (FSARs) for these sites and from the three ESP applications.

Keywords

Probabilistic seismic hazard analysis (PSHA)
Seismic design criteria
New nuclear plant deployment

ABSTRACT

Probabilistic seismic hazard analyses are calculated at 28 nuclear plant sites in the Central and Eastern United States for ground motions with spectral frequencies between 100 Hz and 1 Hz. New information on seismic sources in the region is incorporated in the probabilistic estimates, and a new comprehensive model of ground motion (quantifying both aleatory and epistemic uncertainty) is used. These seismic hazard results quantify the means and fractiles of spectral accelerations with annual frequencies of exceedance between 10^{-3} and 10^{-7}. Results also are calculated as uniform hazard spectra. These hazard results can be used to determine appropriate seismic design criteria for new plants.

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1 INTRODUCTION

This report examines seismic hazard at 28 nuclear plant sites in the Central and Eastern United States (CEUS). It builds upon seismic hazard results reported by EPRI (1989), updating those results to account for new information regarding earthquake occurrences and the associated ground motions.

The 28 sites investigated here constitute a majority of the 29 sites used to establish a reference probability in Regulatory Guide 1.165 (USNRC, 1997). (The Callaway site is not included in this study.) Plants are founded on hard rock, soft rock, and soil of varying thickness and stiffness. Descriptions of site foundation materials are given in Section 3, and site-response calculations are described in Section 5.

In a separate study, these seismic hazard calculations are used to examine the seismic design recommendations for nuclear plants made by ASCE (2005).

2 SEISMIC HAZARD INPUTS

2.1 EPRI Seismic Sources

The seismic hazard calculations conducted here build on the calculations made for the EPRI-Seismicity Owners Group (SOG) study of seismic hazard at nuclear sites in the CEUS (EPRI, 1989). Those calculations used seismic source inputs specified by six Earth Science Teams (ESTs), and used three ground motion equations to calculate the mean and fractiles of seismic hazard at 57 nuclear plant sites. Site-specific reports for each of the 57 nuclear plant sites specify the seismic sources and source combinations used to calculate seismic hazard in the 1989 study. An additional resource used to replicate the assumptions of the 1989 study was the documentation by Risk Engineering, Inc (1989).

2.2 Changes and Additional Seismic Sources

For seismic sources, significant new information has become available on the occurrence of large earthquakes in the CEUS. Three Early Site Permit (ESP) applications (Dominion, 2003; Entergy, 2003; Exelon, 2003) have been submitted to the US Nuclear Regulatory Commission in recent years. All three studies used the EPRI-SOG study as a basis and examined seismicity in the CEUS and determined how the EPRI-SOG sources should be updated to reflect more recent information.

Changes to seismic sources developed in the EPRI-SOG study are concentrated in five regions:

1. Charleston seismic zone. This source of a large historical earthquake on the East Coast in 1886 was modeled with exponential magnitude distributions by the six EPRI-SOG ESTs, with large earthquakes (**M**~6.8 to 7.3) having a recurrence interval of several thousand years. More recent information indicates a mean recurrence interval of about 550 years for the same magnitude event. Further, an East Coast fault system has been hypothesized for the Charleston region and farther north into North Carolina, although this structure is given a low probability of existence and a low probability of activity if it exists outside of South Carolina (Dominion, 2003). This fault system is modeled with two additional East Coast faults, following Dominion (2003). The shorter recurrence interval for large earthquakes in the Charleston seismic zone is modeled with an additional East Coast fault, in addition to the Charleston sources defined by the EPRI-SOG teams, following the Dominion (2003) ESP application.

2. New Madrid seismic zone. This source of three large historical earthquakes in the Central US during 1811-1812 was modeled by the six EPRI-SOG ESTs using exponential magnitude

distributions with activity rates estimated from lower-level seismicity. The recurrence interval of the largest earthquakes (**M**~7.5 to 8.2) was estimated to be several thousand years. More recent evidence indicates a mean recurrence interval of about 500 years for these large earthquakes. They were modeled with additional faults in the New Madrid seismic zone: the Blytheville Arch fault, the East Prairie fault, and the Reelfoot rift fault. A cluster model was used to represent the occurrence of multiple earthquakes on separate faults, as happened over a period of three months in 1811-1812. The cluster model represented the possibility of two or three events occurring within a short period of time, with a mean recurrence (of the cluster) of about 500 years. These three faults were used in addition to the New Madrid seismic zone specified by each of the six EPRI-SOG ESTs, following the model described in Exelon (2003, Appendix B, Section 4.1.1).

3. Wabash Valley and Illinois regions. The seismicity north of the New Madrid seismic zone was modeled by each of the EPRI-SOG ESTs, using a variety of seismic sources. Studies of paleo-earthquake evidence indicate that moderate-to-large earthquakes have occurred in this region in prehistoric times; therefore, the maximum magnitudes of EPRI-SOG team sources were revised upward to reflect this new evidence. In sources representing the Wabash Valley-Southern Illinois region, maximum magnitudes in the range **M**~7.3 to 7.5 were used. For the Central Illinois region, maximum magnitudes in the range **M**~6.3 to 7 were used. For both regions, the maximum magnitude distributions described in Exelon (2003, Appendix B, Sections 4.1.2 and 4.1.3) were adopted.

4. Saline River source. Several lineaments in Southern Arkansas prompted Entergy (2003) to define a seismic source southwest of the New Madrid seismic zone that may have the potential to produce **M**=6 to 7 earthquakes, based on paleoliquefaction and other evidence. Mean recurrence intervals for these earthquakes are estimated to be between 1000 years and 125,000 years, depending on the earthquake magnitude. This source was modeled as an area source.

5. Gulf Coast region. Many seismic sources in the Gulf Coast had maximum magnitude distributions assigned by EPRI-SOG teams that extended below m_b=5.0. The Entergy (2003) study reviewed these sources and revised the maximum magnitude distributions, using a minimum M_{max} value of 5.0 (corresponding to $m_{b,\ max}$~5.4) (Entergy, 2003, Section 2.5.2, page 2.5-49).

Several inconsistencies among assumptions in the three ESP applications were addressed. Two of the three ESP applications used a minimum magnitude for seismic hazard calculations of m_b=5.0 (following EPRI, 1989), and the third used a minimum magnitude of **M**=5.0 (which corresponds to m_b~5.4). For this study we followed the assumption of two of the three ESP applications and adopted a minimum magnitude of m_b=5.0. The Exelon (2003) study used the cluster model to describe earthquake occurrences on the New Madrid faults; the Entergy (2003) study assumed that earthquake occurrences on each fault were independent. This study adopted the cluster model as being more representative of the current understanding of earthquake occurrences in the New Madrid seismic zone.

2.3 Ground Motion Equations

The ground motion equations used in this study are the ones developed by the Electric Power Research Institute (EPRI, 2004) specifically for the CEUS. These consist of estimates of mean log spectral acceleration for 7 structural frequencies (100, 25, 10, 5, 2.5, 1, and 0.5 Hz) and estimates of logarithmic standard deviation. Epistemic uncertainties in both the mean log spectral acceleration and in the logarithmic standard deviation are represented with alternative models, each with an assigned weight. Different models are recommended based on whether the source of earthquakes is in the Mid-Continent region or the Gulf Coast region, whether the source is a general-area source or a non-general-area source, whether the source represents a rifted or non-rifted tectonic feature, and whether the source is modeled with a point or an extended rupture.

3
SITES STUDIED

3.1 Overview

Twenty-eight sites were studied in this project, those being the majority of sites examined in Reg. Guide 1.165 (USNRC, 1997). (A twenty-ninth site studied in USNRC, 1997, the Callaway site, was not studied here because it was not included in the 1989 EPRI study results.) Table 3-1 lists the 28 sites studied in this project and the site category designated in the USNRC (1997) study and in the EPRI (1989) study.

Twelve of the 28 sites were designated as rock sites by both the USNRC (1997) and EPRI (1989) studies, so these were treated as rock sites here, with no site-specific calculations. Sixteen of the 28 sites were designated as some category of soil by either the USNRC (1997) study, the EPRI (1989) study, or both. For some sites, the USNRC (1997) study indicated rock plus a soil category at sites where critical facilities are founded on both. Soil categories used in the USNRC (1997) study are as follows:

- Sand-S1 increasing V_s with depth 25 to 80 feet
- Sand-S2 increasing V_s with depth 80 to 180 feet
- Sand-S3 increasing V_s with depth 180 to 300 feet
- Till-S1 constant V_s with depth 25 to 80 feet
- Till-S2 constant V_s with depth 80 to 180 feet
- Till-S3 constant V_s with depth 180 to 300 feet
- Deep soil all soils >300 feet

where V_s is shear-wave velocity. Soil categories used in the EPRI (1989) study are as follows:

- I 10-30 feet
- II 30-80 feet
- III 80-180 feet
- IV 180-400 feet
- V >400 feet

Table 3-1
28 Plant Sites and Assigned Site Categories

Plant site	EPRI site category	NRC site category	Comments
Beaver Valley	Soil-III	Sand-S1	Site-specific calculation, see Section 4
Bellefonte	Rock	Rock	Treated as rock site
Braidwood	Rock	Rock	Treated as rock site
Brunswick	Soil-III	Sand-S1	Site-specific calculation, see Section 4
Byron	Rock	Rock	Treated as rock site
Catawba	Rock	Rock/Sand-S1	Site-specific calculation, see Section 4
Clinton	Soil-IV	Till-T3	Site-specific calculation, see Section 4
Comanche Peak	Rock	Rock	Treated as rock site
Davis Besse	Rock	Rock	Treated as rock site
Grand Gulf	N/A*	Deep soil	Site-specific calculation, see Section 4
Hope Creek	Soil-V	Deep soil	Site-specific calculation, see Section 4
LaSalle	Soil-III	Till-T2	Site-specific calculation, see Section 4
Limerick	Rock	Rock	Treated as rock site
McGuire	Rock	Rock	Treated as rock site
Millstone	Rock	Rock	Treated as rock site
Nine Mile Point	Rock	Rock/Sand-S1	Site-specific calculation, see Section 4
North Anna	Rock	Rock/Sand-S1	Site-specific calculation, see Section 4
Perry	Rock	Rock	Treated as rock site
River Bend	Site-specific soil	Deep soil	Site-specific calculation, see Section 4
Seabrook	Rock	Rock	Treated as rock site
Shearon Harris	Rock	Sand-S1	Site-specific calculation, see Section 4
South Texas	Site-specific soil	Deep soil	Site-specific calculation, see Section 4
Summer	Rock	Rock/Sand-S1	Site-specific calculation, see Section 4
Three Mile Island	Rock	Rock/Sand-S1	Site-specific calculation, see Section 4
Vogtle	Soil-V	Deep soil	Site-specific calculation, see Section 4
Waterford	Site-specific soil	Deep soil	Site-specific calculation, see Section 4
Watts Bar	Rock	Rock	Treated as rock site
Wolf Creek	Rock	Rock	Treated as rock site

* Grand Gulf not included in published EPRI (1989) results, site studied later.

Figure 3-1 shows a map with the 28 sites, with a key that designates how each site was treated (rock site or site-specific calculation)

Figure 3-1
Map Showing 28 Plant Sites in the CEUS

4
ROCK HAZARD CALCULATIONS

4.1 Seismic Sources Used for Each Site

Seismic hazard calculations were done for rock conditions at each site. The EPRI-SOG seismic sources were used to calculate rock seismic hazard, as explained in Section 2 above. Some sites had additional sources added to reflect the current understanding of earthquake sources, also described in Section 2 above.

Appendix A documents the seismic sources used in the calculation of seismic hazard at each site.

4.2 Verification Studies at Four Sites with EPRI Results

Verification studies were conducted at four sites to verify that the computer code used in this project (FRISK88) accurately replicates the results obtained in the EPRI (1989) study given the same inputs. For these calculations, the original EPRI (1989) sources and ground motion equations were used without modification. The four sites were selected in four parts of the CEUS to replicate seismic hazard for four different regions. These sites were as follows:

- Northeast site
- Mid-Atlantic site
- Southeast site
- Midwest site

Tables 4-1 through 4-4 compare the seismic hazard (annual frequency of exceedence) for four peak ground acceleration (PGA) levels for the results published in the EPRI (1989) study, the replicated results in this study, and the percent difference (difference in the replicated results, compared to the original results).

Table 4-1
Northeast Site: Verification of 1989 Rock Hazard Results

PGA	Percent Difference		
(g)	Mean	Median	85th
0.05	0.08%	-3.06%	14.55%
0.10	0.18%	0.48%	2.51%
0.25	1.04%	2.00%	-1.85%
0.50	2.41%	0.84%	-8.00%

Table 4-2
Mid-Atlantic Site: Verification of 1989 Rock Hazard Results

PGA	Percent Difference		
(g)	Mean	Median	85th
0.05	-1.61%	0.05%	3.70%
0.10	-1.18%	-2.08%	4.35%
0.25	2.69%	2.07%	3.30%
0.50	6.50%	9.53%	-6.57%

Table 4-3
Southeast Site: Verification of 1989 Rock Hazard Results

PGA	Percent Difference		
(g)	Mean	Median	85th
0.05	0.94%	-1.61%	-8.39%
0.10	1.48%	1.05%	-11.71%
0.25	2.22%	4.97%	-3.51%
0.50	3.69%	3.95%	2.15%

Table 4-4
Midwest Site: Verification of 1989 Rock Hazard Results

PGA	Percent Difference		
(g)	Mean	Median	85th
0.05	-0.24%	-4.56%	-2.56%
0.10	-1.39%	-11.94%	-8.67%
0.25	1.17%	0.92%	-11.07%
0.50	6.12%	-5.11%	3.44%

Generally the results in Tables 4-1 through 4-4 show replication of the original results to within several percent, with a few results (generally the 85%) showing a difference of 12% to 15%. A 3% difference in the annual frequency of exceedence corresponds to approximately a 1% difference in ground motion for a given annual frequency of exceedence. Results were compared to PGA levels only, because the EPRI (1989) study reports mean results only for PGA, not for spectral amplitude. Results are available from EPRI (1989) only to two significant figures, which itself implies a precision of ±5% (e.g. an annual frequency of 1.049E-5 would be reported as 1.0E-5, and an annual frequency of 1.050E-5 would be reported as 1.1E-5). One site used for verification is a deep soil site, and rock hazard results were obtained for verification purposes from archived electronic files rather than from EPRI (1989), which only reported soil hazard results.

4.3 Verification Studies at Three Sites with ESP Application Results

At three sites (Clinton, Grand Gulf, and North Anna), owners have submitted ESP applications for new plant construction, and these applications include seismic hazard results. Comparisons were made to published results for these three sites to verify that the current study replicates the seismic hazard obtained in those three site applications. Site-specific calculations of ground motion were made for all three sites, and comparisons were made for rock conditions using available rock results reported for each of the three sites.

Figures 4-1 and 4-2 show rock results reported for the Clinton site (Exelon, 2003), compared to results calculated in this study for rock conditions. The comparisons in Figures 4-1 and 4-2 check the EPRI-SOG seismic sources, the changes to those sources (see Section 2 above), the additional sources used for the New Madrid seismic zone (see Section 2 above), and the EPRI (2004) rock ground motion equations. Figure 4-1 replicates the median, 5%, and 95% seismic hazard for PGA, 5 Hz spectral acceleration (SA), and 1 Hz SA. There is some mismatch for the 5% fractile at annual frequencies of exceedence below 10^{-5} (the current study's results are low compared to the published results), but for the median and 95% hazard the current study accurately replicates the published Clinton results. Figure 4-2 shows that the current study accurately replicates the mean uniform hazard spectra for 10^{-4} and 10^{-5} annual frequencies of exceedence.

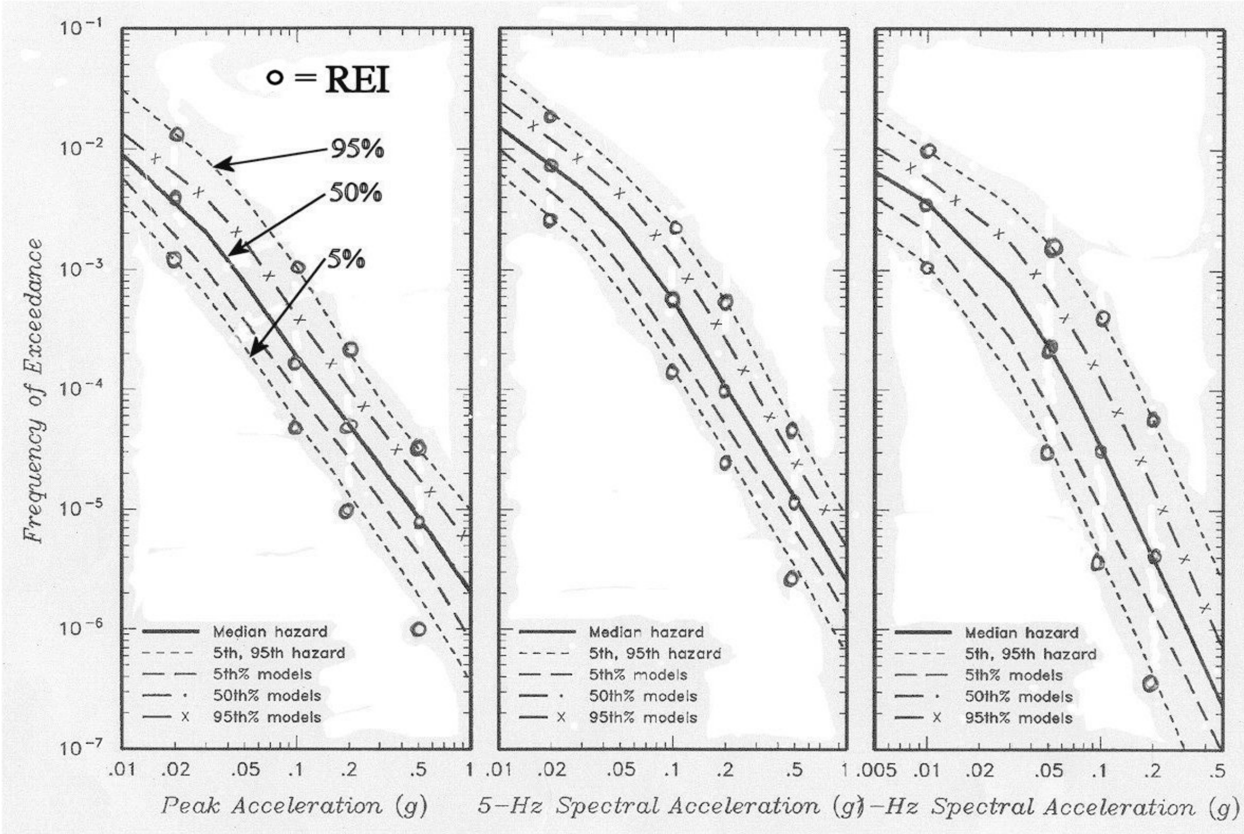

Figure 4-1
Verification of PGA, 5 Hz, and 1 Hz Rock Hazard Results for the Clinton Site (Published Results from Exelon (2003) Site Safety Analysis Report, Appendix B, Figure 4.1-12a)

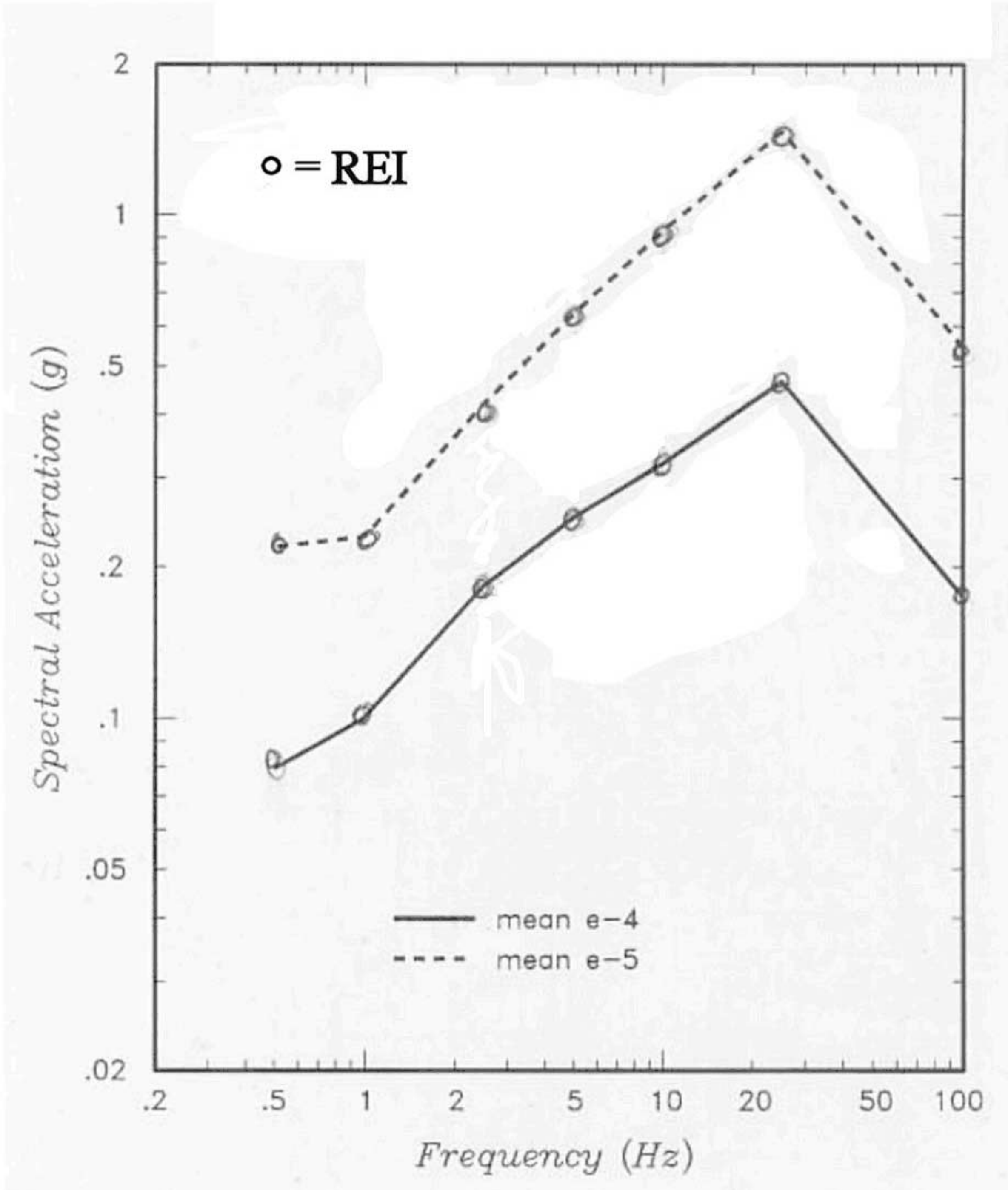

Figure 4-2
Verification of UHS Rock Results for the Clinton Site (Published Results from Exelon (2003) Site Safety Analysis Report, Appendix B, Figure 4.1-19)

Figures 4-3 and 4-4 show rock results reported for the Grand Gulf site (Entergy, 2003), compared to results calculated in this study for rock conditions. Figure 4-3 shows mean, median, 15%, and 85% seismic hazard results for 10 Hz SA, and Figure 4-4 shows a similar comparison for 1 Hz SA. Overall the comparison is excellent, with some mismatch between reported results for the 15% seismic hazard for 10 Hz SA and annual frequencies below 10^{-6} (the current study's results are high compared to the published results). For this comparison the seismic sources were modeled as reported in the Entergy (2003) report; i.e. a minimum magnitude of **M**=5 was used, maximum magnitudes in Gulf Coast sources were modified, and seismic sources representing the New Madrid and Saline River seismic zones were modeled as area sources at the closest approach to the Grand Gulf site.

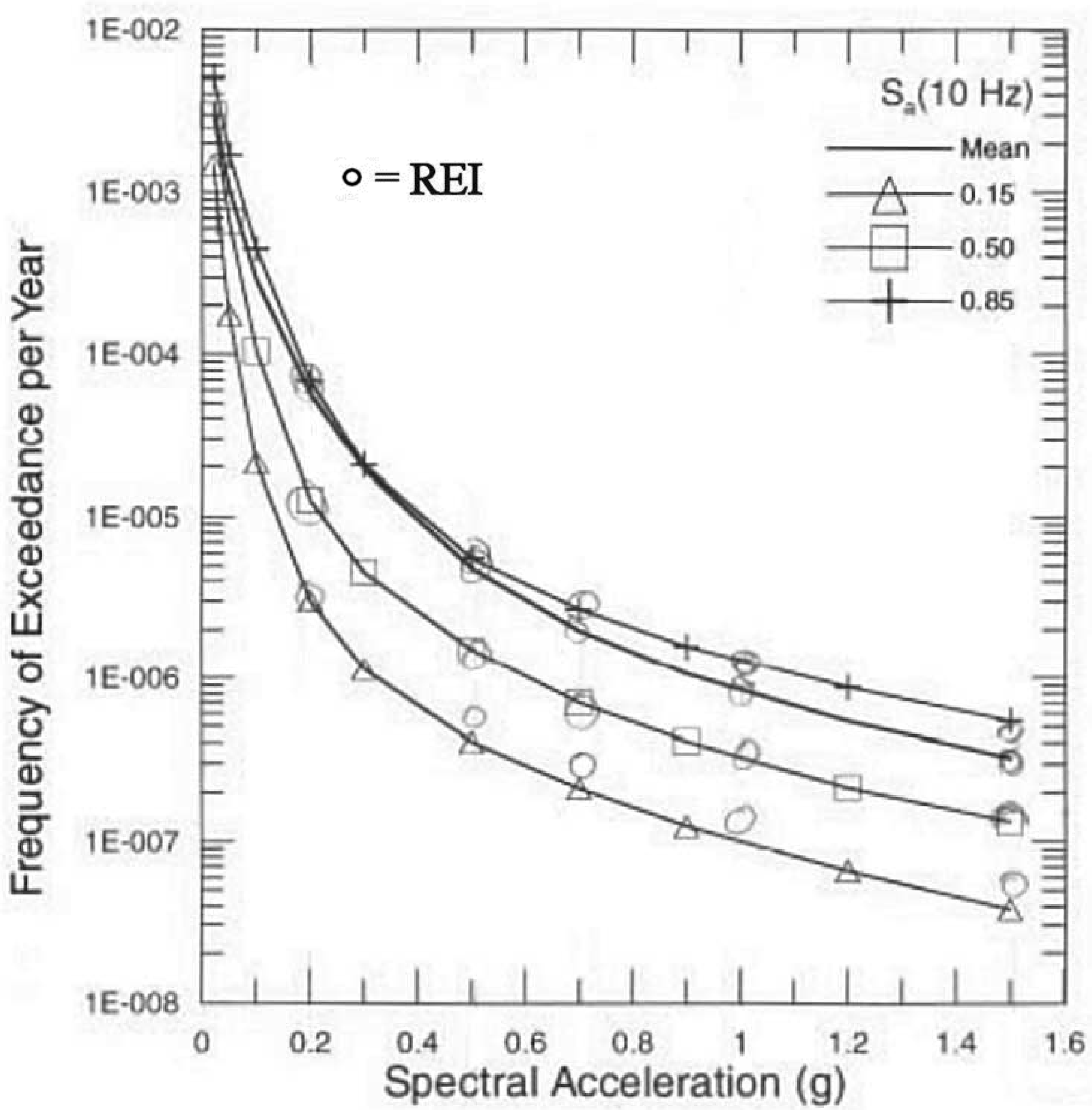

Figure 4-3
Verification of 10 Hz Rock Hazard Results for the Grand Gulf Site (Published Results from Figure 2.5-52 of Entergy, 2003)

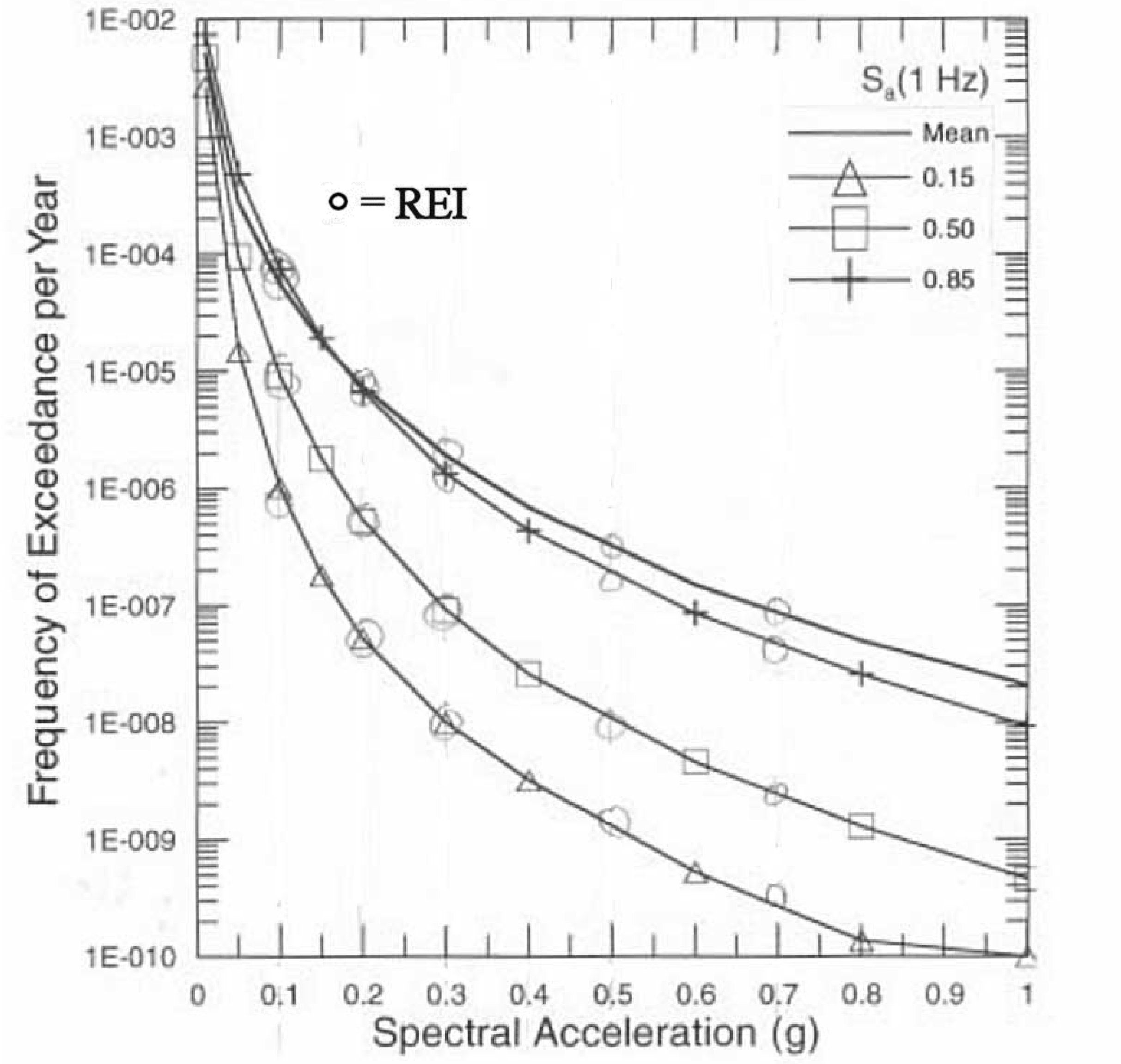

Figure 4-4
Verification of 1 Hz Rock Hazard Results for the Grand Gulf Site (Published Results from Figure 2.5-49 of Entergy, 2003)

Table 4-5 compares ground motions reported for hard rock conditions for the North Anna site (Dominion, 2003) with those calculated in this study, for spectral frequencies from 0.5 Hz to PGA and for annual frequencies of 10^{-4} and 10^{-5}. The same models and seismic hazard software (FRISK88) were used for both studies, so the results are identical. This conclusion applies also to fractiles of seismic hazard for the North Anna site.

Table 4-5
Replication of North Anna Hard Rock Hazard Results

	2003 Reported results*		Replicated results			
Frequency, Hz	**mean 10^{-4} SA, g**	**mean 10^{-5} SA, g**	**mean 10^{-4} SA, g**	**mean 10^{-5} SA, g**	**% difference**	**% difference**
0.5	0.0298	0.0944	0.0298	0.0944	0	0
1	0.0463	0.134	0.0463	0.134	0	0
2.5	0.120	0.364	0.120	0.364	0	0
5	0.235	0.735	0.235	0.735	0	0
10	0.373	1.216	0.373	1.216	0	0
25	0.569	1.99	0.569	1.99	0	0
100 (PGA)	0.214	0.753	0.214	0.753	0	0

*Results taken from Table 2.5-26 of Dominion (2003)

4.4 Rock Hazard Results for 28 Sites

Seismic hazard results were calculated for hard rock conditions for the 28 sites studied under this project. These calculations included the following:

- Seismic hazard curves (mean, 15%, median, and 85%) from 10^{-3} to 10^{-7} annual frequency of exceedence, for six structural frequencies.
- UHS amplitudes (mean, 15%, median, and 85%) for 10^{-4} and 10^{-5} annual frequencies of exceedence.
- Mean SA for 5 Hz and 10 Hz for ten annual frequencies of exceedence, ranging from $5x10^{-4}$ to $5x10^{-7}$.

5
DEVELOPMENT OF SITE SPECIFIC AMPLIFICATION FACTORS

Site specific equivalent-linear site response analyses were performed for each of the 16 sites listed in Table 3-1 as having a site-specific calculation. The foundation levels of the reactor buildings were used to assess the thickness of surficial materials at the sites. For all the sites considered, where soils extended to depths exceeding 500 ft or the shear-wave velocity exceeded about 3,500 ft/sec (1,067m/sec), linear response was assumed (Silva et al.; 1997, 1998a, 1998b, 1999, 2000).

To develop the amplification factors, control motions were specified at the surface of the Mid-continent crustal model (EPRI, 1993) with a shear-wave velocity of 2.83 km/sec, a defined shallow crustal damping parameter (kappa; Anderson and Hough, 1984) of 0.006 sec, and a frequency-dependent deep-crustal damping Q model of 670 $f^{0.33}$ (EPRI, 1993). These values are consistent with the EPRI attenuation models (EPRI, 2004). Distances were then determined to generate a suite of motions with expected peak acceleration values which cover the range of spectral accelerations (0.5, 1.0, 2.5, 5.0, 10.0, 25.0, 100.0 Hz) anticipated at the sites analyzed. To cover the range of motions, 18 expected (median) peak acceleration values were run from 0.01g to 5.00g (Table 5-1). A single moment magnitude **M** 6.5 was used with the stochastic point source model (Silva et al.; 1999, 2000). This magnitude reflects a reasonable average over the sites, structural frequencies, and hazard levels considered. Amplification factors were then developed by placing the site profile on the Mid-Continent crustal model at each distance, generating soil motions, and taking the ratios of soil response spectra to rock response spectra (both at 5% damping). For the higher levels of rock motions, above about 1 to 2g for the softer profiles, the high-frequency amplification factors were significantly less than 1, which may be exaggerated. To adjust the factors for these cases a heuristic lower bound of 0.5 was implemented (EPRI, 1993; Abrahamson and Silva, 1997).

Table 5-1
Distances and Depths Used to Generate Hard Rock Peak Accelerations

Expected Peak Acceleration (%g)	Epicentral Distance (km)	Depth (km)
1	235.0	8.0
5	80.0	8.0
10	47.0	8.0
20	26.0	8.0
30	18.0	8.0
40	13.0	8.0
50	9.5	8.0
75	3.0	8.0
100	0.0	6.5
125	0.0	5.3
150	0.0	4.5
200	0.0	3.3
250	0.0	2.7
300	0.0	2.3
350	0.0	1.9
400	0.0	1.7
450	0.0	1.5
500	0.0	1.4

To accommodate aleatory variability in dynamic material properties expected to occur across each site (footprint), shear-wave velocity profiles, G/G_{max}, and hysteretic damping curves were randomized. Since depth to basement material (defined as shear-wave velocity of 2.83 km/sec (9,285 ft/sec)) is poorly known at many deep soil sites, it was taken at a large enough depth to easily accommodate maximum soil amplification to the lowest frequency of interest, 0.5 Hz (Silva et al., 1999, 2000). For these cases, basement depth was randomized over a large range as well, to smooth over potential low-frequency resonances. For sites where depth to basement is relatively well known, a more restrictive range was used. In all cases, the basement depth randomization assumed a uniform distribution (EPRI, 1993).

For these deep soil and sedimentary rock sites, shear-wave velocities are poorly known for depths below those characterized by the site investigations. The approach taken here was to assume a deep base-case profile based on shear-wave velocity measurements made in similar materials and depths (analog profiles). Epistemic variability in the deep velocities was accommodated by considering higher and lower mean deep-velocity profiles, all with the same shallow profile, based on site investigations. As is well known in site response analyses, alternative reasonable assumptions regarding deep velocities beneath soil sites has little impact on computed amplification, provided the site total kappa value (Anderson and Hough, 1984) remains fixed. Kappa, at low strain, controls the high frequency ($\geq$ 5 Hz) amplification and an effort has been made to provide reasonably conservative yet realistic estimates of base-case values for each site, based on experience with similar sites that have measured values (Anderson and Hough, 1984; Silva and Darragh, 1995; Silva et al., 1997). Epistemic variability in kappa is accommodated by considering higher and lower values, generally with about a 50% variation on the base-case values. Naturally, as loading levels increase and non-linearity becomes more pronounced, the potential impacts of the assumed kappa values decreases.

The profile randomization scheme, which varies both layer velocity and thickness, was based on a correlation model developed from an analysis of variance of about 500 measured shear-wave velocity profiles (EPRI, 1993; Silva et al., 1997). This model used variability in velocity that was appropriate for a large structural footprint. The parametric variation includes profile velocity layer thickness, depth to basement material (2.83 km/sec), G/G_{max}, and hysteretic damping curves.

To accommodate variability in the modulus reduction and damping curves on a generic basis, the curves were independently randomized about the base case values. A lognormal distribution was assumed with a logarithmic Φ of 0.30 at a cyclic shear strain of 3×10^{-2}% with upper and lower bounds of 2Φ (EPRI, 1993). The distribution was based on an analysis of variance of measured G/G_{max} and hysteretic damping curves and was considered appropriate for applications to generic (material type specific) nonlinear properties. The truncation was necessary to prevent modulus reduction or damping models that were not physically realizable. The random curves were generated by sampling the transformed normal distribution with a logarithmic Φ of 0.35, computing the change in normalized modulus reduction or percent damping at 3×10^{-2}% shear strain, and applying this factor at all strains. The random perturbation factor was reduced or tapered near the ends of the strain range to preserve the general shape of the median curves (Silva, 1992; EPRI, 1993).

To accommodate epistemic variability in dynamic material properties, multiple base case (mean) models were considered, each with associated aleatory variability captured by the randomization process. Amplification factors were then expressed as median and $\pm 1\sigma$ estimates based on 100 realizations at each distance (Silva et al., 1999; 2000). Median amplification factors reflecting the site epistemic variability were then used to develop soil hazard curves from the rock hazard curves using the analytical approximation recommended in NUREG/CR-6769 (Equation A-16 from REI, 2002) and Bazzurro and Cornell (Equation 17 from Bazzurro and Cornell, 2004). The epistemic variability in the rock hazard was preserved by using a distribution of rock hazard reflected in twenty fractiles evenly distributed from the 2.5 to 97.5 percentiles. Site epistemic variability was captured through multiple soil hazard curves corresponding to each rock hazard fractile. For cases where the analytical estimate of the soil hazard was invalid, the more accurate approach of conditioning the rock hazard with the probability distribution of the site-specific amplification factors was used (Bazzurro and Cornell, 2004)

6 CONCLUSIONS

This study applies state-of-the-art seismic hazard software to calculate seismic hazard at 28 nuclear plant sites in the CEUS. The basis for the seismic sources used in these hazard calculations is the set of sources developed and documented during the EPRI (1989) study. These sources are updated using interpretations published in more recent applications. The ground motion equations used here are those developed and published by EPRI (2004).

Verification of calculations made here with results published by EPRI (1989), using the same inputs, show that similar results are obtained (generally within several percent, in terms of annual frequency of exceedence for a given ground motion). This verifies that the software used in this study (the FRISK88 package) accurately calculates seismic hazard. Additional verification is made by comparing the FRISK88 results with rock hazard results documented in three ESP applications submitted by utilities in 2003. This verification shows that the updates to the seismic sources, and the EPRI (2004) ground motion equations, are accurately represented and calculated.

Quantitative comparisons between the current results and those published for the 28 sites in the EPRI (1989) study have not been made. Nevertheless, qualitative conclusions can be drawn regarding the effects of new information.

The interpretations of seismic sources in the New Madrid and Charleston regions indicates that current estimates of the possible sizes of large earthquakes in those regions are similar to those made in 1989. However, current estimates of recurrence intervals are shorter than those used in 1989, and this leads to higher seismic hazard at sites affected by those sources, particularly for lower-frequency ground motions. Recent studies in Southern and Central Illinois indicate that moderate-to-large earthquakes might be possible for sources located there, and these moderate-to-large earthquakes were not modeled in the 1989 study. The result is that estimates of seismic hazard are higher for sites in Illinois. In the Gulf Coast, seismic activity rates are low and the possible range of magnitudes is limited to small and moderate earthquakes, but estimates of possible earthquake size have increased somewhat. This increases seismic hazard slightly for sites located in the Gulf Coast, but estimates of ground motion hazard remain low.

7
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A

SEISMIC SOURCES USED IN THE CALCULATIONS FOR EACH OF THE 28 SITES

Table A-1
Seismic Sources used for the Beaver Valley Site

TEAM	SEISMIC SOURCES
BECHTEL	24 25A BZ5 BZ6
DAMES & MOORE	04 4C 07 08 73
LAW ENGINEERING	17 112
RONDOUT	12 C02
WESTON GEOPHYSICAL	101 102 C12 C13 C14 C15 C16 C19
WOODWARD-CLYDE	35 61 63 B69

Table A-2
Seismic Sources used for the Bellefonte Site

TEAM	SEISMIC SOURCES
BECHTEL	25 25A 30 BZ0 BZ3 BZ5 BZ6
DAMES & MOORE	08 21 41 54 71
LAW ENGINEERING	17 18 115
RONDOUT	1 9 13 25 26 C02
WESTON GEOPHYSICAL	24 31 32 C11 C17 C19
WOODWARD-CLYDE	29 29A 31A 40 B39
(ADDITIONAL)	3 NEW MADRID FAULTS
(ADDITIONAL)	EAST COAST FAULT SYSTEM - SOUTH

Table A-3
Seismic Sources used for the Braidwood Site

TEAM	SEISMIC SOURCES					
BECHTEL	30	BZ3				
DAMES & MOORE	15A	16B	18A	21	70	71
LAW ENGINEERING	18	116				
RONDOUT	1	4	15	52		
WESTON GEOPHYSICAL	30	31	33	105	C29	
WOODWARD-CLYDE	36	36A	44	56	B62	
(ADDITIONAL)	3 NEW MADRID FAULTS					

Table A-4
Seismic Sources used for the Brunswick Site

TEAM	SEISMIC SOURCES							
BECHTEL	H	N3	BZ4	BZ5	C07			
DAMES & MOORE	53	54						
LAW ENGINEERING	22	35	107	108	C09	C10	C11	M35
RONDOUT	24	26	C01					
WESTON GEOPHYSICAL	25	26	104	C20	C21	C23	C24	C26
	C27	C33	C35					
WOODWARD-CLYDE	29	29A	29B	30	B23			
(ADDITIONAL)	3 EAST COAST FAULTS							

Table A-5
Seismic Sources used for the Byron Site

TEAM	SEISMIC SOURCES
BECHTEL	BZ3
DAMES & MOORE	15A 16B 17 18A 70 71
LAW ENGINEERING	116
RONDOUT	15
WESTON GEOPHYSICAL	30 100 105 C29
WOODWARD-CLYDE	55 56 B61
(ADDITIONAL)	3 NEW MADRID FAULTS

Table A-6
Seismic Sources used for the Catawba Site

TEAM	SEISMIC SOURCES
BECHTEL	F G H N3 BZ4 BZ5
DAMES & MOORE	04 4B 41 53 54
LAW ENGINEERING	17 22 107 108 C09 C10 C11 M31 M32 M33 M34 M35 M36 M37
RONDOUT	24 25 26 27 28
WESTON GEOPHYSICAL	24 25 26 104 C17 C19 C20 C21 C23 C24 C26 C27 C33 C35
WOODWARD-CLYDE	29 29A 29B 30 31A B28
(ADDITIONAL)	3 EAST COAST FAULTS

Table A-7
Seismic Sources used for the Clinton Site

TEAM	SEISMIC SOURCES
BECHTEL	30 BZ0 BZ3 K
DAMES & MOORE	18 18A 19 21 70 71
LAW ENGINEERING	06 07 18 116
RONDOUT	1 2 4 15 52
WESTON GEOPHYSICAL	31 32 33 34 105 C11 C29
WOODWARD-CLYDE	40 42 43 44 B47
(ADDITIONAL)	3 NEW MADRID FAULTS

Table A-8
Seismic Sources used for the Comanche Peak Site

TEAM	SEISMIC SOURCES
BECHTEL	38 39 BZ2 BZ3 C04
DAMES & MOORE	20 25 25A 28 28B 67
LAW ENGINEERING	26 124
RONDOUT	16 C02
WESTON GEOPHYSICAL	36 109 C31
WOODWARD-CLYDE	46 46A B44
(ADDITIONAL)	3 NEW MADRID FAULTS

Table A-9
Seismic Sources used for the David Besse Site

TEAM	SEISMIC SOURCES							
BECHTEL	N1	BZ3	BZ6	C10				
DAMES & MOORE	07	08	12	14	14B	15	70	73
LAW ENGINEERING	111	112	115					
RONDOUT	7	8	10	11	12	C02		
WESTON GEOPHYSICAL	29	101	105	C13	C14	C16		
WOODWARD-CLYDE	35	36	37	38	39	B68		

Table A-10
Seismic Sources used for the Grand Gulf Site

TEAM	SEISMIC SOURCES				
BECHTEL	30	BZ1	BZ3		
DAMES & MOORE	20	21	25	C15	
LAW ENGINEERING	18	126			
RONDOUT	1	16	51		
WESTON GEOPHYSICAL	31	32	36	107	C11
WOODWARD-CLYDE	40	44	B40		
(ADDITIONAL)	3 NEW MADRID FAULTS				
(ADDITIONAL)	SALINE RIVER SOURCE				

Table A-11
Seismic Sources used for the Hope Creek Site

TEAM	SEISMIC SOURCES							
BECHTEL	BZ4	BZ5						
DAMES & MOORE	04	4D	41	42	47	53		
LAW ENGINEERING	17	22	107	C09	C10	C11	C13	M16
	M17	M18	M19	M20	M21			
RONDOUT	30	31						
WESTON GEOPHYSICAL	28A	C08	C09	C22	C23	C28	C34	
WOODWARD-CLYDE	21	21A	22	23	24	53	63	B09

Table A-12
Seismic Sources used for the LaSalle Site

TEAM	SEISMIC SOURCES					
BECHTEL	30	BZ3				
DAMES & MOORE	15A	16B	18A	21	70	71
LAW ENGINEERING	18	116				
RONDOUT	1	4	15			
WESTON GEOPHYSICAL	30	31	105	C29		
WOODWARD-CLYDE	44	56	B60			
(ADDITIONAL)	3 NEW MADRID FAULTS					

Table A-13
Seismic Sources used for the Limerick Site

TEAM	SEISMIC SOURCES
BECHTEL	13 BZ4 BZ5
DAMES & MOORE	04 4D 08 41 42 47 53
LAW ENGINEERING	17 22 217 C09 C10 M14 M15 M16 M17 M18 M19
RONDOUT	30 31
WESTON GEOPHYSICAL	28A 28B C07 C10 C19 C22 C23 C28 C34
WOODWARD-CLYDE	21 21A 22 23 24 53 63 B18

Table A-14
Seismic Sources used for the McGuire Site

TEAM	SEISMIC SOURCES
BECHTEL	F G H N3 BZ4 BZ5
DAMES & MOORE	04 4B 41 53 54
LAW ENGINEERING	17 22 35 107 217 C10 C11 M31 M32 M33 M34 M35 M36
RONDOUT	24 25 26 27 28 C02
WESTON GEOPHYSICAL	23 24 25 26 C17 C18 C19 C20 C21 C23 C24 C27 C33
WOODWARD-CLYDE	29 29A 29B 30 31 31A B26
(ADDITIONAL)	3 EAST COAST FAULTS

Table A-15
Seismic Sources used for the Millstone Site

TEAM	SEISMIC SOURCES							
BECHTEL	08	B	BZ4	BZ5	BZ8			
DAMES & MOORE	2	4	4A	41	47	53	63	C14
LAW ENGINEERING	17	22	102	103	C09	M10	M11	M12
	M13	M14	M15					
RONDOUT	31	40	41	44				
WESTON GEOPHYSICAL	06	10	13	16	17	19	20	28A
	39	C06	C07	C10				
WOODWARD-CLYDE	08	10	11	23	57	59	B07	

Table A-16
Seismic Sources used for the Nine Mile Point Site

TEAM	SEISMIC SOURCES							
BECHTEL	07	11	C	D	BZ5	BZ6	BZ7	C05
DAMES & MOORE	03	09	38	39B	C02	C09	C10	C11
LAW ENGINEERING	11	17	111					
RONDOUT	33	34	35	47	C02			
WESTON GEOPHYSICAL	04	05	07	08	C12	C13	C14	C16
WOODWARD-CLYDE	15	18	19	33	34	C10	B14	

Table A-17
Seismic Sources used for the North Anna Site

TEAM	SEISMIC SOURCES							
BECHTEL	24	E	BZ4	BZ5				
DAMES & MOORE	04	4B	40	41	42	47	53	
LAW ENGINEERING	17	22	107	217	C09	C10	C11	M19
	M20	M21	M22	M23	M24	M27		
RONDOUT	28	29	30					
WESTON GEOPHYSICAL	22	C19	C21	C22	C23	C34	C35	
WOODWARD-CLYDE	26	27	29	29A	B22			
(ADDITIONAL)	EAST COAST FAULT SYSTEM - SOUTH							

Table A-18
Seismic Sources used for the Perry Site

TEAM	SEISMIC SOURCES						
BECHTEL	27	D	N1	BZ6	C06		
DAMES & MOORE	07	08	14	14B	15	70	73
LAW ENGINEERING	111	112					
RONDOUT	10	11	12	33	C02		
WESTON GEOPHYSICAL	07	101	C12	C14	C15	C16	C32
WOODWARD-CLYDE	33	35	61	63	B70		

Table A-19
Seismic Sources used for the River Bend Site

TEAM	SEISMIC SOURCES			
BECHTEL	30	BZ1		
DAMES & MOORE	20	21	25	C15
LAW ENGINEERING	18	126		
RONDOUT	1	51		
WESTON GEOPHYSICAL	31	107		
WOODWARD-CLYDE	40	B42		
(ADDITIONAL)	3 NEW MADRID FAULTS			
(ADDITIONAL)	SALINE RIVER SOURCE			

Table A-20
Seismic Sources used for the Seabrook Site

TEAM	SEISMIC SOURCES							
BECHTEL	03	08	09	B	BZ4	BZ7	BZ8	
DAMES & MOORE	02	53	56	59	61	63		
LAW ENGINEERING	12	21	22	24	102	103	C09	C12
	M08	M09	M10					
RONDOUT	31	37	40	41	43			
WESTON GEOPHYSICAL	01	13	14	16	17	C03	C04	C05
WOODWARD-CLYDE	06	08	09	12	58	59	B02	

Table A-21
Seismic Sources used for the Shearon Harris Site

TEAM	SEISMIC SOURCES							
BECHTEL	13	F	H	N3	BZ4	BZ5		
DAMES & MOORE	40	41	53	54				
LAW ENGINEERING	17	35	107	C09	C10	C11	M27	M28
	M31	M32	M33	M34	M35			
RONDOUT	24	26	28	29	C01			
WESTON GEOPHYSICAL	22	25	26	28D	104	C19	C20	C21
	C22	C24	C25	C26	C28	C33	C35	
WOODWARD-CLYDE	26	27	29	29A	29B	30	B24	
(ADDITIONAL)	3 EAST COAST FAULTS							

Table A-22
Seismic Sources used for the South Texas Site

TEAM	SEISMIC SOURCES	
BECHTEL	BZ1	BZ2
DAMES & MOORE	20	25
LAW ENGINEERING	124	126
RONDOUT	51	
WESTON GEOPHYSICAL	107	
WOODWARD-CLYDE	B43	
(ADDITIONAL)	3 NEW MADRID FAULTS	
(ADDITIONAL)	SALINE RIVER SOURCE	

Table A-23
Seismic Sources used for the Summer Site

TEAM	SEISMIC SOURCES
BECHTEL	F G H N3 BZ4 BZ5
DAMES & MOORE	41 53 54
LAW ENGINEERING	17 22 107 108 C09 C10 C11 M31 M32 M33 M34 M36 M37 M38 M39
RONDOUT	24 26
WESTON GEOPHYSICAL	25 26 104 C19 C20 C21 C23 C24 C26 C27 C33 C35
WOODWARD-CLYDE	29 29A 29B 30 31A B31
(ADDITIONAL)	3 EAST COAST FAULTS

Table A-24
Seismic Sources used for the Three Mile Island Site

TEAM	SEISMIC SOURCES
BECHTEL	13 24 BZ4 BZ5
DAMES & MOORE	04 4C 08 41 42 47
LAW ENGINEERING	17 22 217 C09 C10 C11 M16 M17 M18 M19 N20 M21
RONDOUT	30 31
WESTON GEOPHYSICAL	28A 28B 102 C08 C09 C17 C18 C22 C23 C28 C34
WOODWARD-CLYDE	21 21A 22 23 53 61 63 B17

Table A-25
Seismic Sources used for the Vogtle Site

TEAM	SEISMIC SOURCES
BECHTEL	F G H N3 BZ4 BZ5
DAMES & MOORE	20 41 52 53 54
LAW ENGINEERING	17 22 35 108 C09 C10 C11 M33 M36 M37 M38 M39 M40 M41 M42
RONDOUT	24 26
WESTON GEOPHYSICAL	25 26 104 C19 C20 C21 C23 C24 C26 C27 C33 C35
WOODWARD-CLYDE	29 29A 29B 30 B32
(ADDITIONAL)	3 EAST COAST FAULTS

Table A-26
Seismic Sources used for the Waterford Site

TEAM	SEISMIC SOURCES
BECHTEL	30 BZ1
DAMES & MOORE	21 20
LAW ENGINEERING	18 126
RONDOUT	1 51
WESTON GEOPHYSICAL	31 32 107 C11
WOODWARD-CLYDE	40 B41
(ADDITIONAL)	3 NEW MADRID FAULTS
(ADDITIONAL)	SALINE RIVER SOURCE

Table A-27
Seismic Sources used for the Watts Bar Site

TEAM	SEISMIC SOURCES
BECHTEL	24 25 25A 30 F H BZ0 BZ5 BZ6
DAMES & MOORE	04 4A 05 21 41 54
LAW ENGINEERING	01 17 18 115 217
RONDOUT	1 5 9 25 26 27
WESTON GEOPHYSICAL	24 31 C17 C19
WOODWARD-CLYDE	29 29A 29B 31 31A 40 B29
(ADDITIONAL)	3 NEW MADRID FAULTS
(ADDITIONAL)	EAST COAST FAULT SYSTEM - SOUTH

Table A-28
Seismic Sources used for the Wolf Creek Site

TEAM	SEISMIC SOURCES
BECHTEL	41 42 BZ3 C01 C02 C03
DAMES & MOORE	35B 36 37 37B 68A 68B 69
LAW ENGINEERING	30 118 119 C04 C05
RONDOUT	18 21
WESTON GEOPHYSICAL	35 108 C30
WOODWARD-CLYDE	45 47 54 60 B48
(ADDITIONAL)	3 NEW MADRID FAULTS

B
SITE DESCRIPTIONS FOR SITE-SPECIFIC ANALYSES

This Appendix contains descriptions of site conditions for the 16 sites that were handled with site-specific calculations. Several sites use the following empirical relation between kappa and the average shear-wave velocity at rock sites (Silva et al., 1997).

$$\log(\text{kappa}) = 2.2189 - 1.0930 * \log(V_S), \qquad \textbf{Eq. B-1}$$

Multiple models are used to capture epistemic uncertainty in site response, and these models (and their weights) are described in the subsection for each of the 16 sites.

B.1 BEAVER VALLEY SITE

The Beaver Valley Power Station is located along the Ohio River a few miles (several km) east of the Pennsylvania-Ohio border. The site is within the Appalachian Plateau physiographic province.

The site is underlain by older Pleistocene (?) terrace deposits (sand and gravel, containing variable amounts of cobbles and rock fragments) within an erosional bedrock valley of the Ohio River. The terrace has a maximum thickness of about 100 ft (30.5m) and rests directly upon bedrock of Pennsylvanian age. The bedrock underlying the site consists primarily of Paleozoic sandstone and shale with inter-bedded coal units. Below the Paleozoic sandstones and coal seams lie shales, sandstones, and siltstones as well as the Salina Group which contains salt beds. The uppermost salt bed occurs at a depth of about 4,700 ft (1,432m) below the plant site. Precambrian crystalline basement rock was estimated to be at a depth of about 10,500 ft (3,200m) (EPRI, 1989).

B.1.1 Soil Profile Information

The reactor containment building is founded on the terrace deposits with an embedment depth of 54 ft (16.5m). The terrace deposits are about 46 ft (14.0m) thick beneath this structure. Bedrock of Paleozoic age underlies the terrace deposits. Basement rock is estimated to be at a depth of 5,000 ft (1,524m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Weston Geophysical made seismic wave velocity measurements into the bedrock at 100 ft (30.5m). In the terrace deposits the shear wave velocity ranged from 600 to 1,200 f/sec (182.9 to 365.7 m/sec) above the water table and 1,300 ft/sec (396.2 m/sec) below the water table. The depth to the water table was measured between 66 and 69 ft (20 and 21m). The measured shear-wave velocity at the top of the bedrock (shale) is 6,000 ft/sec (1,828.7 m/sec).

Site-specific measurements of modulus reduction and damping curves were not available in REI (1989).

B.1.2 Description of Base Case Profiles

Discussed below are the base case site dynamic material properties intended to capture site epistemic variability (uncertainty). Aleatory variability (randomness) is accommodated through randomization about the base case properties. Multiple median (logarithmic mean) amplification factors (over aleatory variability) are then weighted to accommodate site epistemic variability in the site-specific soil hazard curves.

B.1.2.1 Shear-Wave Velocity Profiles

Measured shear-wave velocities extend to a depth of about 50 ft below the reactor containment structure in Figure B-1 and are estimated from velocities in similar material beyond this depth. The stair-stepped profile (M1P1 in Figure B-1) is considered the base-case profile and is continued to a depth of 5,000 ft (1,524m) (randomized ± 2,000 ft) (610m). Lower and higher velocity alternatives (M1P2 and M1P3 respectively) are considered as well, with M1P3 reaching hard rock shear-wave velocities (9,285 ft/sec, 2.83 km/sec) at a depth of about 2,000 ft (610m) (randomized ± 500 ft (152m)).

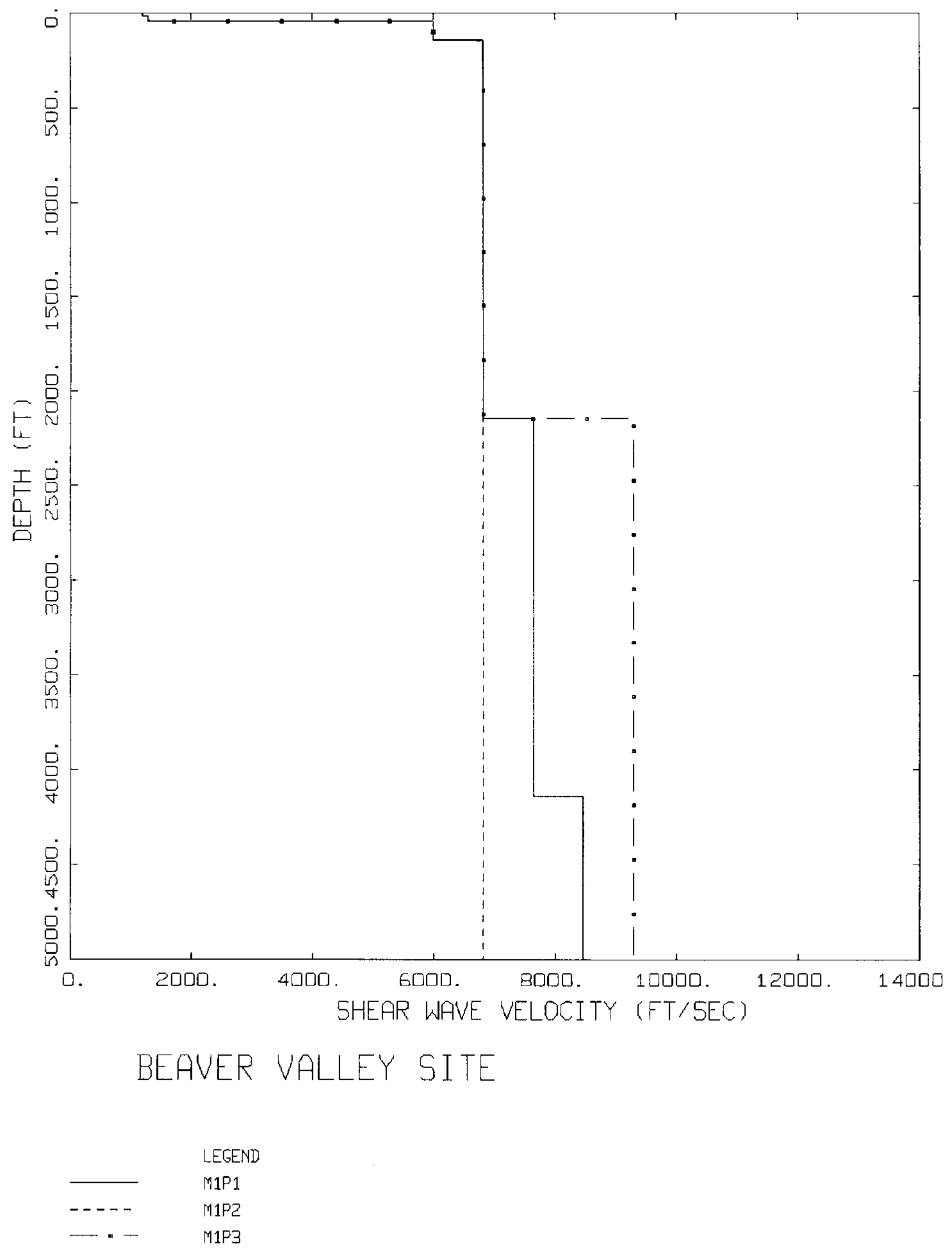

Figure B-1
Shear-Wave Velocity Profiles for the Beaver Valley Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate lower and higher at-depth velocities. All three profiles are estimates below a depth of about 50 ft (15m).

B.1.2.2 Modulus Reduction and Hysteretic Damping Curves

For the terrace sands and gravels in the top 50 ft (15m) of the profile, the EPRI (1993) cohesionless soil curves are considered appropriate. The materials with shear-wave velocities of 6,000 ft/sec (1,829 m/sec) and greater are considered linear.

B.1.2.3 Regional Crustal Damping (kappa)

Based on Equation B-1, the kappa value for 6,000 ft/sec (1,828.7 m/sec) material is 0.0123 sec, Adding the low strain damping from the EPRI (1993) curves for the approximately 50 ft of gravely soil below the reactor containment vessel, a kappa value of about 0.0013 sec, the total base-case kappa at the soil surface is about 0.0136 sec. This value is assumed appropriate for the base-case profile (M1P1) as well as the low and high velocity profiles (M1P2 and M1P3, Figure B-1). To consider alternative mean kappa values for the base case profile, a high total kappa (M1P1.KH) of 0.04 sec, a typical value for soft rock in Western North America was assumed. For a low kappa (M1P1.KL) a rock (6,000 ft/sec, 1,828.71 m/sec) value of 0.01 sec was assumed for a low kappa total site value of 0.0113 sec.

B.1.2.4 Profile Weights

Table B-1
Beaver Valley Weights

Properties*	Category Weights
M1P1	0.5
M1P1.KH	0.2
M1P1.KL	0.3
M1P1	0.6
M1P2	0.2
M1P3	0.2
	Combined Weights
M1P1	0.30
M1P1.KH	0.12
M1P1.KL	0.18
M1P2	0.20
M1P3	0.20

*M1P1; base-case profile, kappa = 0.0136 sec

M1P1.KH; base-case profile, kappa = 0.04 sec

M1P1.KL; base-case profile, kappa = 0.0113 sec

M1P2; low gradient profile, kappa = 0.0136 sec

M1P3; high gradient profile, kappa = 0.0136 sec

B.2 BRUNSWICK SITE

The Brunswick Steam Electric Plant is located in the southeastern portion of North Carolina. The site is located on the Atlantic seaboard within 5 miles (8.0 km) of the Atlantic Ocean and is within the Atlantic Coastal Plain physiographic province. Its location is about 90 miles (145 km) southeast of the Fall Line, the boundary between the flat lying deposits of the Coastal Plain and the folded formations of the Piedmont and Appalachian regions.

The site is underlain by the Miocene Yorktown Formation (alternating clay and sand), Oligocene sediments (limestone over lenses of clay and sand), Eocene Castle Hayne Limestone (shell limestone over sandstone with some clay) and the Cretaceous Peedee Formation (calcareous clay and sand). Crystalline basement rock was estimated to be at a depth of about 1,500 ft (457.2m) (EPRI, 1989).

B.2.1 Soil Profile Information

The reactor buildings are founded on very dense sand of the Yorktown formation at a depth of about 48 ft (14.6m). Prior to construction, this sand was overlain by loose sands (Pamlico Terrace Formation) and soft silty clays and clay silts (top of Yorktown Formation). The engineered fill of the plant island was placed on the lower 30 ft (9.1m) of the Yorktown formation, which is composed of medium to coarse grained and well-compacted sand with minor lenses of clay near the top. The dense sand is underlain by 80 ft (24.4m) of limestone that is underlain by 70 ft (21.3m) of sandstone over 270 ft of silty clay and clayey silt to a depth of 600 ft (182.9m). The bottom silty clay unit (Peedee) is well consolidated, soft to medium hard using a rock hardness classification (EPRI, 1989). Basement rock is estimated to be at a depth of 1,500 ft (457.2m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Weston Geophysical made seismic wave velocity measurements to a depth of 600 ft (182.9m). In the stiff sand of the Yorktown formation, assumed foundation material, the shear-wave velocity was measured as 1,400 ft/sec (426.7 m/sec). The measured shear-wave velocity at the top of the limestone is 5,500 ft/sec (1,676.3 m/sec). Below the limestone the measured shear-wave velocity decreases to 4,500 ft/sec (1,371.5 m/sec) in sandstone and to 3,000 ft/sec (914.4 m/sec) in the Peedee formation with 600 ft (183m) reflecting the deepest measurements.

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site (EPRI, 1989).

B.2.2 Description of Base Case Profiles

B.2.2.1 Shear Wave Velocity Profiles

Figure B-2 shows the measured shear-wave velocity (to a depth of about 600 ft (182.9m)) with the deepest measurements extrapolated to crystalline basement at an estimated depth of 1,500 ft (randomized + 500 ft) (457 + 152m). This is considered the base case profile (M1P1, Figure B-

2) and profile M1P2 (Figure B-2) is intended to accommodate the potential for a velocity gradient in the stiff silty clay. The reactor building is founded at a depth of 48 ft (14.6m), which was removed from the measured shear-wave velocities.

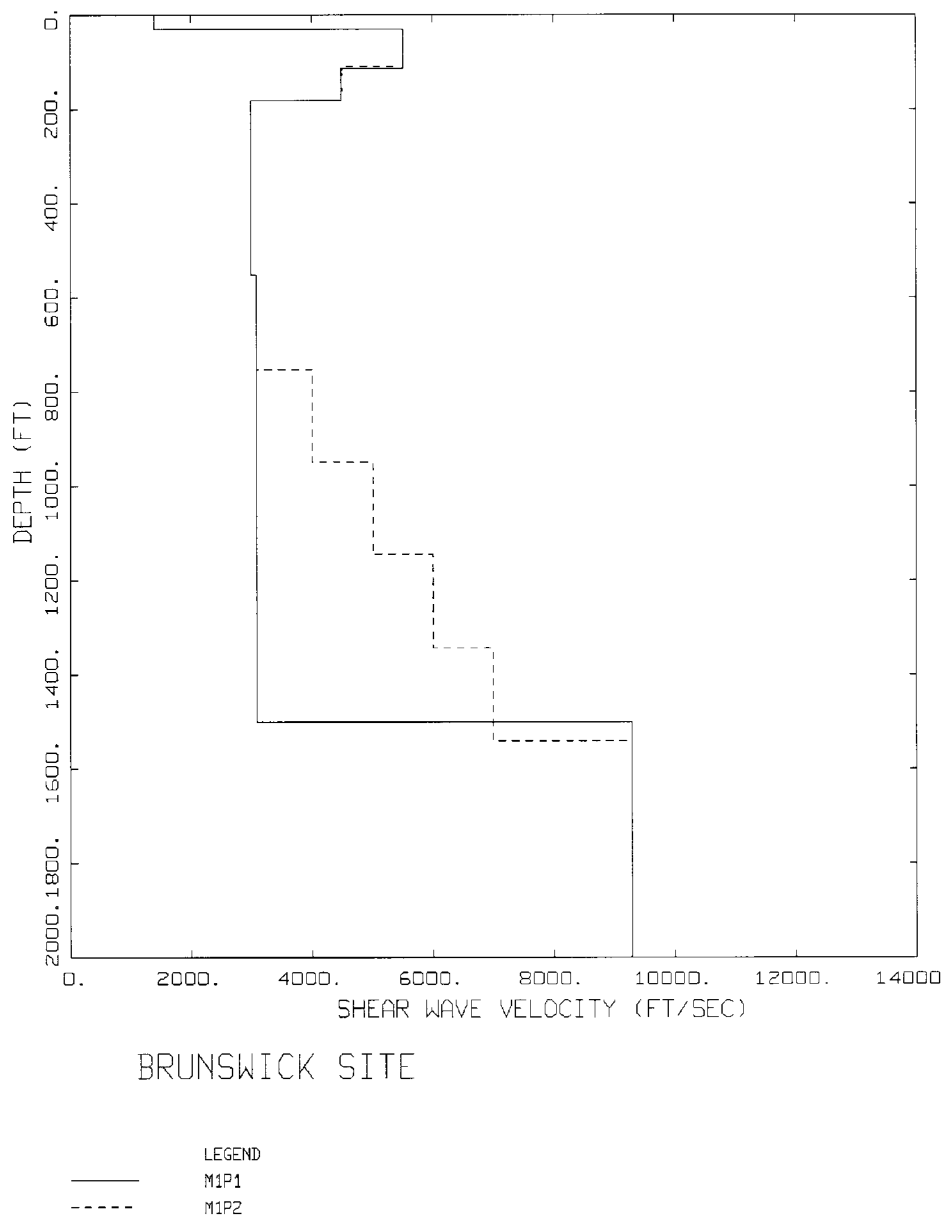

Figure B-2
Shear-Wave Velocity Profiles for the Brunswick Site

Profile M1P1 is considered the base case profile with M1P2 to accommodate a gradient in the deep stiff silty clay. Both profiles are estimates below a depth of about 600 ft (183m).

B.2.2.2 Modulus Reduction and Hysteretic Damping Curves

For the units which have similar soil types and depth ranges, G/G_{max} and hysteretic damping curves from the well characterized Savannah River Site, located within the Atlantic Coastal Plain along the Georgia and South Carolina border, were used. The Dry Branch curves from Savannah River Site (SRS, 1996) were used for the stiff sands of the Miocene Yorktown Formation in the top 30 ft (9.1 m) of the profile. Peninsular Range rock curves (Silva et al., 1997) were used for the limestone and sandstone. For the silty clays of the Peedee formation, the deep clay curves from the Savannah River Site (SRS, 1996) were selected. The materials are considered to behave linearly below a depth of 500 ft (152.4 m; Silva et al.; 1997, 1998b).

B.2.2.3 Regional Crustal Damping (kappa)

A kappa of 0.02 sec was assumed as the base case value for this site based on the similarity of the profile to the 1,000 ft (304.8m) soil profile of the Savannah River Site, where a value for kappa of 0.02 sec has been measured. To accommodate the possibility of a higher kappa value due to the additional 500 ft of soil, relative to the Savannah River profile, amplification factors were also developed for a kappa value of 0.03 sec. The high gradient profile (M1P2, Figure B-2) was assumed to reflect the base case kappa value of 0.02 sec.

B.2.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-2.

Table B-2
Brunswick Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.4
M1P1	0.8
M1P2	0.2
	Combined Weights
M1P1	0.48
M1P1.KH	0.32
M1P2	0.20

*M1P1; base case profile, kappa = 0.02 sec

M1P1.KH; base case profile, kappa = 0.03 sec

M1P2; high gradient profile, kappa = 0.02 sec

B.3 CATAWBA SITE

The Catawba Nuclear Station is located adjacent to Lake Wylie in York County, in the north central portion of South Carolina. The site is located in the Charlotte Belt of the Piedmont physiographic province; a deeply eroded plateau-like segment of the Appalachian Mountain System. The Charlotte Belt is characterized by an extensive complex of intrusive; and with the exception of a few broad folds is dominated by plutonic contacts.

The site is located on a Paleozoic basement rock consisting of adamellite (predominant rock underlying the site), amphibolite, diorite, porphyritic diorite, aplite and pegmatite. A thin soil and a zone of weathered bedrock (saprolite) overlie fresh unweathered crystalline bedrock, which is encountered at depths of 25 to 75 ft (7.6 to 22.9 m) across the plant area (EPRI, 1989).

B.3.1 Soil Profile Information

The reactor buildings are founded crystalline bedrock at a depth of about 50 ft (15.2 m) below finished grade (0 to 30 ft (0 to 9.1 m) below the original surface) with the central core about 77 ft (23.5m) deep. The foundation excavation required the removal of the overlying soil and weathered rock. Basement rock is estimated to be at a depth of 119 ft (36.3m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical measurements included seismic refraction, uphole, and cross-hole. During the uphole survey, measurements of shear wave velocity were made to a depth of 120 ft (36.6m). Measured shear-wave velocities in the bedrock ranged from 3,000 ft/sec (914.4 m/sec) to 9,000 ft/sec (2,743 m/sec).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site (EPRI, 1989).

B.3.2 Description of Base Case Profiles

B.3.2.1 Shear Wave Velocity Profiles

The base-case shear-wave velocity profile is shown in Figure B-3 (M1P1) and is based on geophysical surveys at the site. It shows a gradient over the top 100 ft (30.47m) increasing from about 3,000 ft/sec (914.36 m/sec) at the surface (average embedment depth) to the reference rock velocity of 9,285 ft/sec (2.83 km/sec) at about 100 ft (30.47m). This softer rock zone is likely an artifact of partial weathering and fracturing and was not completely stripped from the site. To accommodate portions of embedment in contact with hard rock, profile M1P2 (Figure B-3) considers reference rock conditions at the surface (50 to 80 ft, 15 to 24m below actual grade).

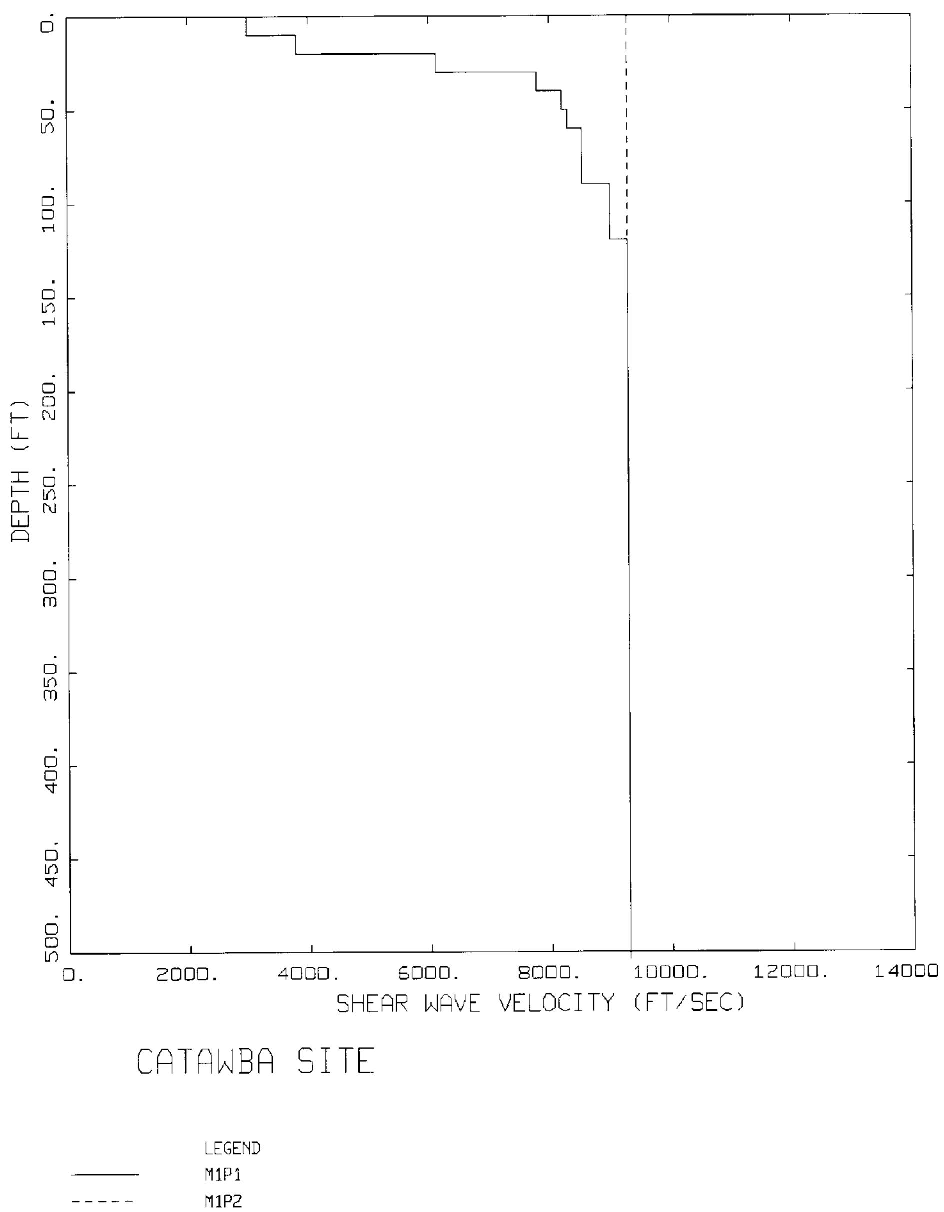

Figure B-3
Shear-Wave Velocity Profiles for the Catawba Site

Profile M1P1 is considered the base case profile and is based on measured velocities to a depth of about 70 ft (21m). Profile M1P2 accommodates the high range of shear-wave velocities of the bedrock and reflects the hard rock reference velocity of 9,285 ft/sec (2.83 km/sec).

B.3.2.2 Modulus Reduction and Hysteretic Damping Curves

For the base case profile (M1P1), Peninsular Range cohesion-less soil curves (Silva et al., 1997) were used for the top 10 ft (3.0m). Peninsular Range rock curves (Silva et al., 1997) were used from 10 ft (3.0m) to 70 ft (21.3m).

B.3.2.3 Regional Crustal Damping (kappa)

For the hard rock below a depth of about 70 ft (21.3m), the CEUS standard value of 0.006 sec was assumed. The kappa contributed by the low strain material in profile M1P1 damping at shallower depths is about 0.001 sec, for a total site kappa value of 0.007 sec. The reference rock profile kappa value is assumed to be 0.006 sec, reflecting hard rock outcrop hazard.

B.3.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-3.

Table B-3
Catawba Weights

Properties*	Category Weights
M1P1	0.6
M1P2	0.4
	Combined Weights
M1P1	0.6
M1P2	0.4

*M1P1; base-case profile, kappa = 0.007 sec

M1P2; low gradient profile, kappa = 0.007 sec

B.4 CLINTON SITE

The Clinton Power Station is located in the Illinois Basin, slightly west of the La Salle Anticlinal Belt about 6 miles (9.7 km) east of the city of Clinton, Illinois. The site is within the Till Plains section of the Central Lowland physiographic province.

Strata underlying the site consist of an estimated 170 to 360 ft (51.8 to 109.7m) of Quaternary overburden, largely Wisconsinan, Illinoian, and pre-Illinoian aged glacial deposits resting on essentially flat-lying Pennsylvanian-aged shales, sandstones and thin coal beds. Precambrian crystalline basement rock was estimated to be at a depth of about 4,000 ft (1,219m) (Exelon, 2003).

B.4.1 Soil Profile Information

Major power block structures are founded on compacted fill resting on stiff Illinoian till at an embedment depth of about 56 ft (17.1m). Basement rock is estimated to be at a depth of 4,000 ft (1,219m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Recent suspension shear-wave velocity measurements (1 hole) were made into the bedrock at a depth about 300 ft (91m) (Exelon, 2003). In the top 42 ft (1.3m) of loess and weathered Wisconsinian glacial till deposits, the shear-wave velocity is 975 ft/sec (297 m/sec). In the next 17 ft (5.2m) of overburden the velocity is 1,343 ft/sec (409 m/sec). In the Illinoian and pre-Illinoian glacial till above the bedrock the shear-wave velocity is about 2,000 ft/sec (609 m/sec). The measured shear-wave velocity at the top of the bedrock (limestone, shale, and sandstone) is about 4,000 ft/sec (1,219 m/sec) at a depth of about 300 ft (91m) (Exelon, 2003).

CH2M Hill performed laboratory testing. Resonant column and torsional shear dynamic tests were performed to estimate site-specific modulus reduction and hysteretic damping curves (Exelon, 2003).

B.4.2 Description of Base Case Profiles

B.4.2.1 Shear Wave Velocity Profiles

The base-case shear-wave velocity profile (M1P1) is shown in Figure B-4 with the top 300 ft (91m) based on a smoothed suspension log survey (Exelon, 2003) which penetrated local bedrock of shales, sandstones and coal beds. Precambrian basement lies at an estimated depth of 4,000 ft (1,219m). For depths below about 300 ft (91m), several regional (within about 10 miles, 16 km) oil well compressional-wave surveys were available with at least one extending to a depth of about 5,000 ft (1,524m). Based on assumptions of values for Poisson’s ratios for these materials (0.25 to 0.35), the base-case profile was extended to a depth of 4,000 ft (1,219m) and randomized ± 2,000 ft (610m). To consider alternative deep velocities, profile M1P2 provides for a shear-wave velocity of nearly 6,000 ft/sec (1,288 m/sec) to Precambrian basement while profile M1P3 considers Precambrian basement velocities occur locally at a depth of 1,200 ft (366m) (randomized ± 400 ft, ± 122m).

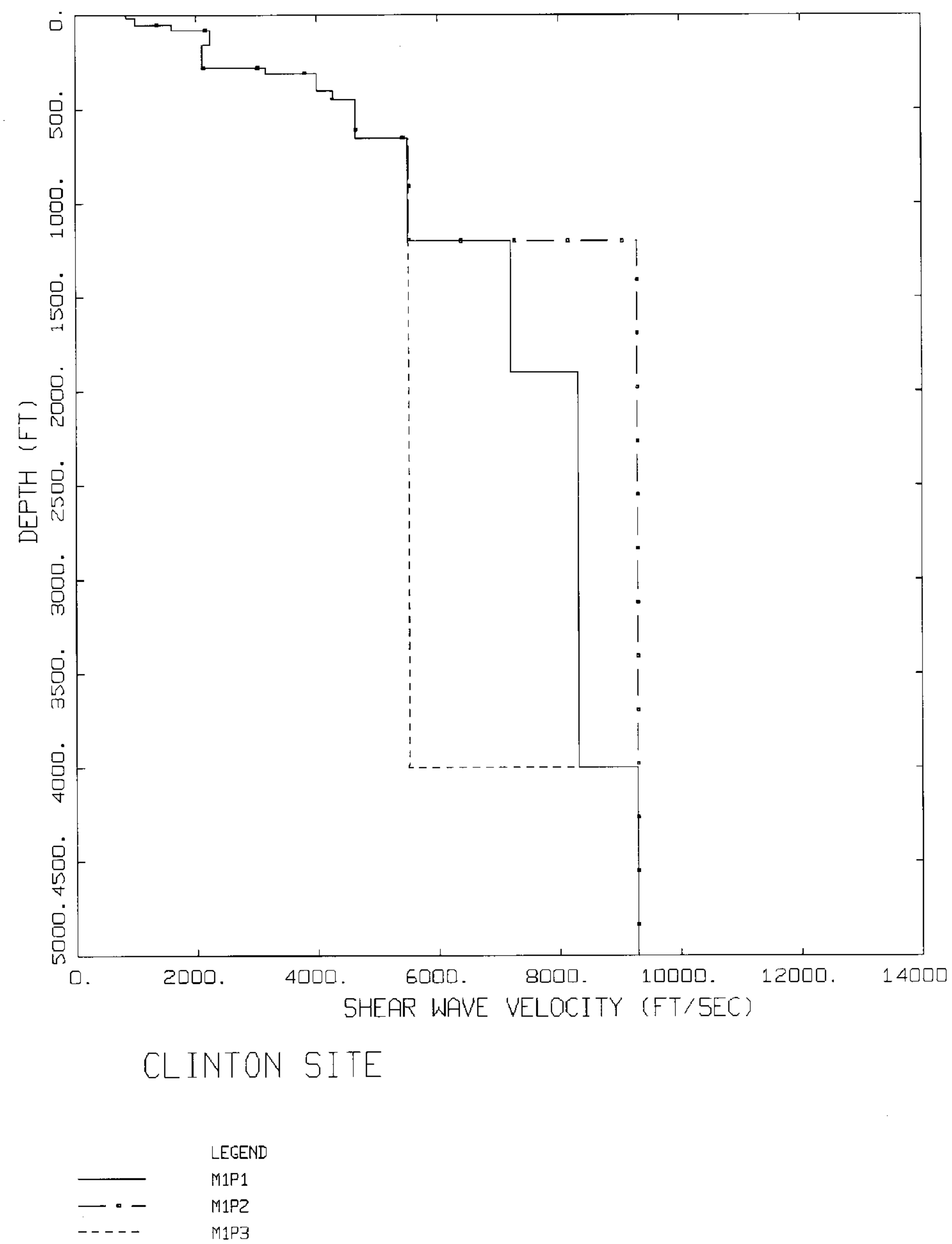

Figure B-4
Shear-Wave Velocity Profiles for the Clinton Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate higher and lower at-depth velocities. All three profiles are estimates below a depth of about 300 ft (91m).

B.4.2.2 Modulus Reduction and Hysteretic Damping Curves

Based on comparison of recent resonant column and torsional shear test results with the EPRI (1993) cohesion-less soil curves, the EPRI curves were adopted for the glacial deposits overlying the shale and sandstone bedrock (Exelon, 2003).

B.4.2.3 Regional Crustal Damping (kappa)

To assess an appropriate kappa value for the site (shallow soil over sedimentary rock), the empirical rock site relation between kappa and the average shear-wave velocity over the top 100 ft (31m) (Equation B-1) was applied to the rock beneath the soil. For a shear-wave velocity of 4,000 ft/sec (1,219 m/sec) (Figure B-4), the estimated kappa value is 0.019 sec. Adding the low strain damping in the soil section, with a kappa value of 0.0034 sec, results in a total site kappa value of 0.0224 sec. To accommodate the possibility that the 300 ft (91m) of soil and nearly 4,000 ft (1,219m) of sedimentary rock may have a kappa value similar to Western North America soft rock, a total kappa value of 0.04 sec (Silva and Darragh, 1995; Silva et al., 1997) was also used. For the deep low (profile M1P2) and high (profile M1P3) velocity profiles (Figure B-4), the base-case total kappa value of 0.0224 sec was used.

B.4.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-4.

Table B-4
Clinton Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.4
M1P1	0.6
M1P2	0.2
M1P3	0.2
	Combined Weights
M1P1	0.36
M1P1.KH	0.40
M1P2	0.12
M1P3	0.12

*M1P1; base case profile, kappa = 0.0224 sec

M1P1.KH; base case profile, kappa = 0.04 sec

M1P2; low gradient profile, kappa = 0.0224 sec

M1P2; high gradient profile, kappa = 0.0224 sec

B.5 GRAND GULF SITE

The Grand Gulf Nuclear Station is located in west-central Mississippi about 25 miles (40 km) south of Vicksburg. The site is within the Loess Hills (Uplands) sub province at the western margin bordering the Mississippi Alluvial Valley (Lowlands) sub province of the Gulf Coastal Plain.

The site is underlain by a sequence of late Pliocene to Quaternary eolian and alluvial deposits overlying the Miocene Catahoula Formation that consists of non-marine and littoral bedrock. The Catahoula bedrock underlying the site consists of weakly cemented claystone that extends to the bottom of the deepest boring (447 ft, 136m). The strata underlying the site consist of a thick and stratigraphically complex sequence of relatively flat lying sediments that are part of the Gulf Coast geosyncline. These sediments are about 20,000 ft (6,000m) thick and unconformably overlie a sequence or rocks composed mainly of Mesozoic limestone. Precambrian crystalline basement rock was estimated to be at a depth of about 27,000 ft (8,200m) (Entergy, 2003).

B.5.1 Soil Profile Information

The Loess deposits were removed at the site and the reactor is founded on alluvium of the Upland Complex. Precambrian basement rock is estimated to be at a depth of 27,000 ft (8,200m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical refraction and crosshole seismic surveys were performed in 1971 to 1972 (Entergy (1994)). Recently, the shear-wave velocity profile at the site was based on three P-S suspension velocity log surveys, with the deepest extending to a depth of about 225 ft (Entergy, 2003). The shallow materials consist of about 75 ft (23m) of loess, 85 ft (26m) of young alluvium, with old alluvium to a depth of about 200 ft (61m) where claystones of the Upland Catahoula formation were encountered. Both the old and young alluvium comprise the terrace deposits of the Uplands. The maximum depth of the suspension log surveys was about 225 ft (69m).

William Lettis and Associates also performed laboratory testing. Resonant column and torsional shear dynamic tests were performed to estimate site-specific modulus reduction and hysteretic damping curves (Entergy, 2003).

B.5.2 Description of Base Case Profiles

B.5.2.1 Shear Wave Velocity Profiles

To extend the measured profile to a depth of about 3,000 ft (914m), a generic Mississippi embayment shear-wave velocity profile was used. This generic profile was developed for ground shaking studies in the embayment by Professor Glenn Rix of the MAE Center (personal communication, 2002). The profile is based on a large number of shallow and several deep velocity surveys and extends to a depth of 3,600 (1,100m). For the site base case profile, the shallow velocities to a depth of about 225 ft replaced those of the generic Mississippi embayment upland profile, which had similar velocities (about 2,000 ft/sec) at these depths. The complete base case profile is shown in Figure B-5 (profile M1P1) to a depth of about 3,200 ft (975m) where shear-wave velocity is set to 2.83 km/sec, appropriate for hard rock conditions. To consider alternative deep shear-wave velocities, profile M1P2 provides for a low sediment velocity to Precambrian basement while profile M1P3 considers the possibility of a rapidly increasing shear-wave velocity with depth. All three profiles have basement depth randomized ± 500 ft (152m).

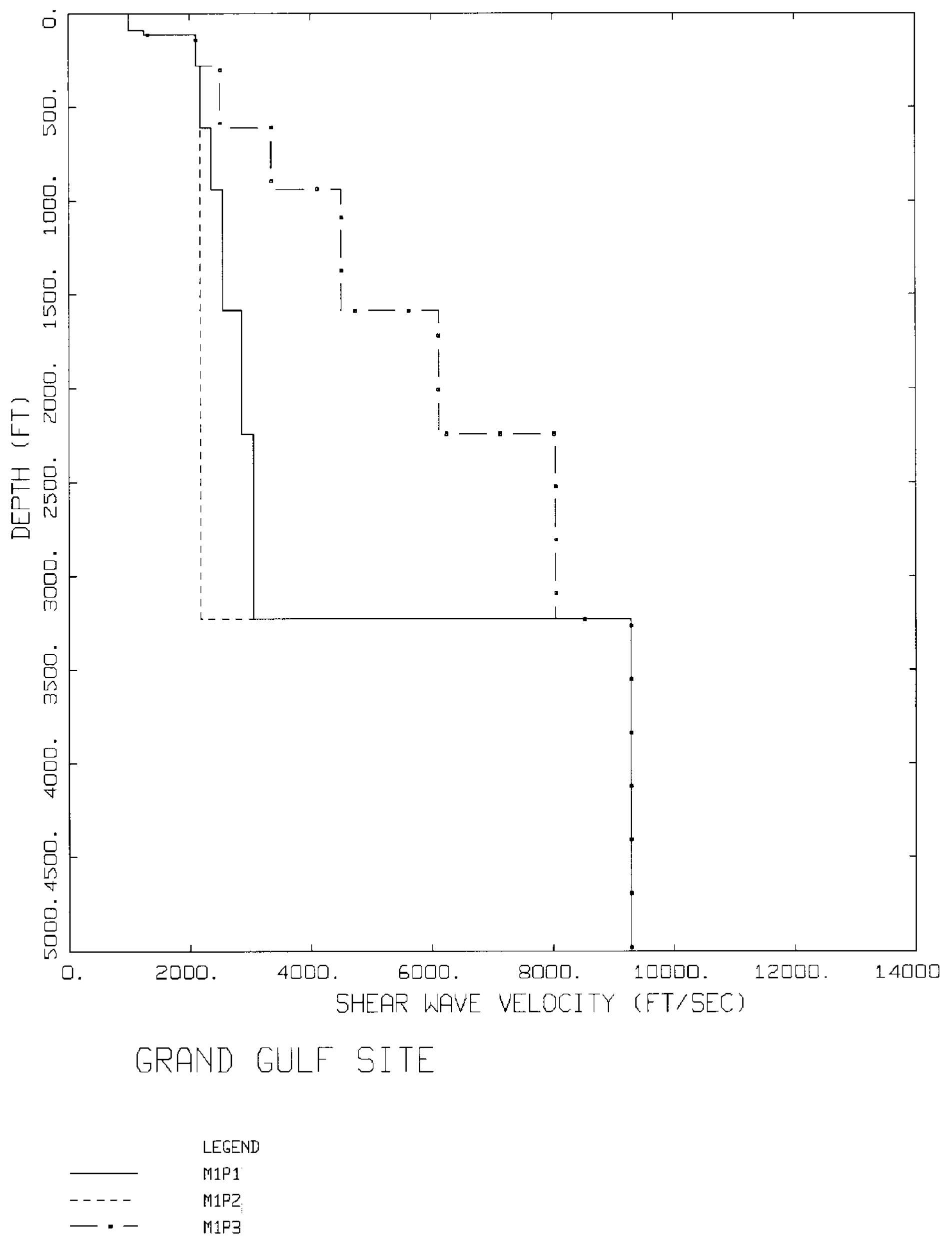

Figure B-5
Shear-Wave Velocity Profiles for the Grand Gulf Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate lower and higher at-depth velocities. All three profiles are estimates below a depth of about 200 ft (61m).

B.5.2.2 Modulus Reduction and Hysteretic Damping Curves

Based on comparison of recent resonant column and torsional shear test results with the EPRI (1993) cohesion-less soil curves, the EPRI curves were adopted for the deposits with a site-specific assignment (Entergy, 2003). For the claystones at depths below the site characterization, the Peninsular Range curves were used, to a depth of 500 ft (152 m) with linear response below (Entergy, 2003).

B.5.2.3 Regional Crustal Damping (kappa)

For the deep sedimentary basin, 27,000 ft (8,200m) to Precambrian basement, a total base-case site kappa value of 0.046 sec was assumed. This is considered a conservative (low) value for this region of the embayment (Professor R. Herrmann personal communication, 2001) and based on deep soils/sediments in the Western United States (Anderson and Hough, 1984; Silva et al., 1997). Alternative considerations for a lower total kappa value of 0.028 sec (M1P1.KH) and a higher total kappa value of 0.066 sec were also used for the base-case profile (M1P1) and are based on subtracting and adding a kappa of about 0.02 sec to the base case value of 0.046 sec.

B.5.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-5.

Table B-5
Grand Gulf Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.2
M1P1.KL	0.2
M1P1	0.8
M1P2	0.1
M1P3	0.1
	Combined Weights
M1P1	0.48
M1P1.KH	0.16
M1P1.KL	0.16
M1P2	0.10
M1P3	0.10

*M1P1; base case profile, kappa = 0.046 sec

M1P1.KH; base case profile, kappa = 0.066 sec

M1P1.KL; base case profile, kappa = 0.028 sec

M1P2; low gradient profile, kappa = 0.046 sec

M1P2; high gradient profile, kappa = 0.046 sec

B.6 HOPE CREEK SITE

The Hope Creek Generating Station is located on an artificial island, a man-made promontory on the east bank of the Delaware River in New Jersey. The site is within the Atlantic Coastal Plain physiographic province about 18 miles (29 km) southeast of the Fall Line.

The site is underlain by three Quaternary units including 30 to 45 ft (9 to14m) of hydraulic fill, 2 to 12 ft (0.6 to 3.7m) of coarse sand and gravel and 5 to 20 ft (1.5 to 6.1m) of non-organic clay. These strata are deposited on the Miocene Kirkwood Formation, a 2 to 6 foot (0.6 to 1.8m) thick basal sand overlying a silty organic clay. The Kirkwood unconformably overlies the Eocene Vincentown Formation that consists of basal sandstone and two overlying sand units. The soils above and into the Vincentown were removed to a depth of approximately that 72 ft (22m) at the location of the power block (EPRI, 1989).

The Vincentown conformably overlies 14 to 20 ft (4.3 to 6.1m) of fine-to-medium sand and silt of the Paleocene Hornerstown Formation. The Mesozoic strata are primarily sands with clay and gravel. Precambrian and Early Paleozoic crystalline basement rock was estimated to be at a depth of about 1,800 ft (550m).

B.6.1 Soil Profile Information

The Quaternary units including the fill as well as the Miocene Kirkwood Formation were removed during the construction of the power block. The power block is founded on the sands of the Vincentown Formation. Basement rock is estimated to be at a depth of 1,800 ft (550m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical refraction and up-hole seismic studies were performed to measure seismic velocities at the site to a depth of 400 ft (122m). The average shear-wave velocity is 1,850 ft/sec (564 m/sec) in the Vincentown and Hornerstown Formations and the underlying strata.

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site (EPRI, 1989).

B.6.2 Description of Base Case Profiles

B.6.2.1 Shear Wave Velocity Profiles

Figure B-6 shows the base-case shear-wave velocity profile (M1P1) with an assumed increase in velocity from depth of 400 ft (122m) to 800 ft (244m) where the shear-wave velocity is taken to reach 3,000 ft/sec (914 m/sec), due to age and confinement. Alternatively, the velocity at a depth of about 400 ft (122m) (1,850 ft/sec, 564 m/sec), based on up-hole measurements, is continued to Precambrian basement at a depth of 1,800 ft (550m) (profile M1P2, Figure B-6). Both profiles are randomized in depth $\pm$500 ft (152m).

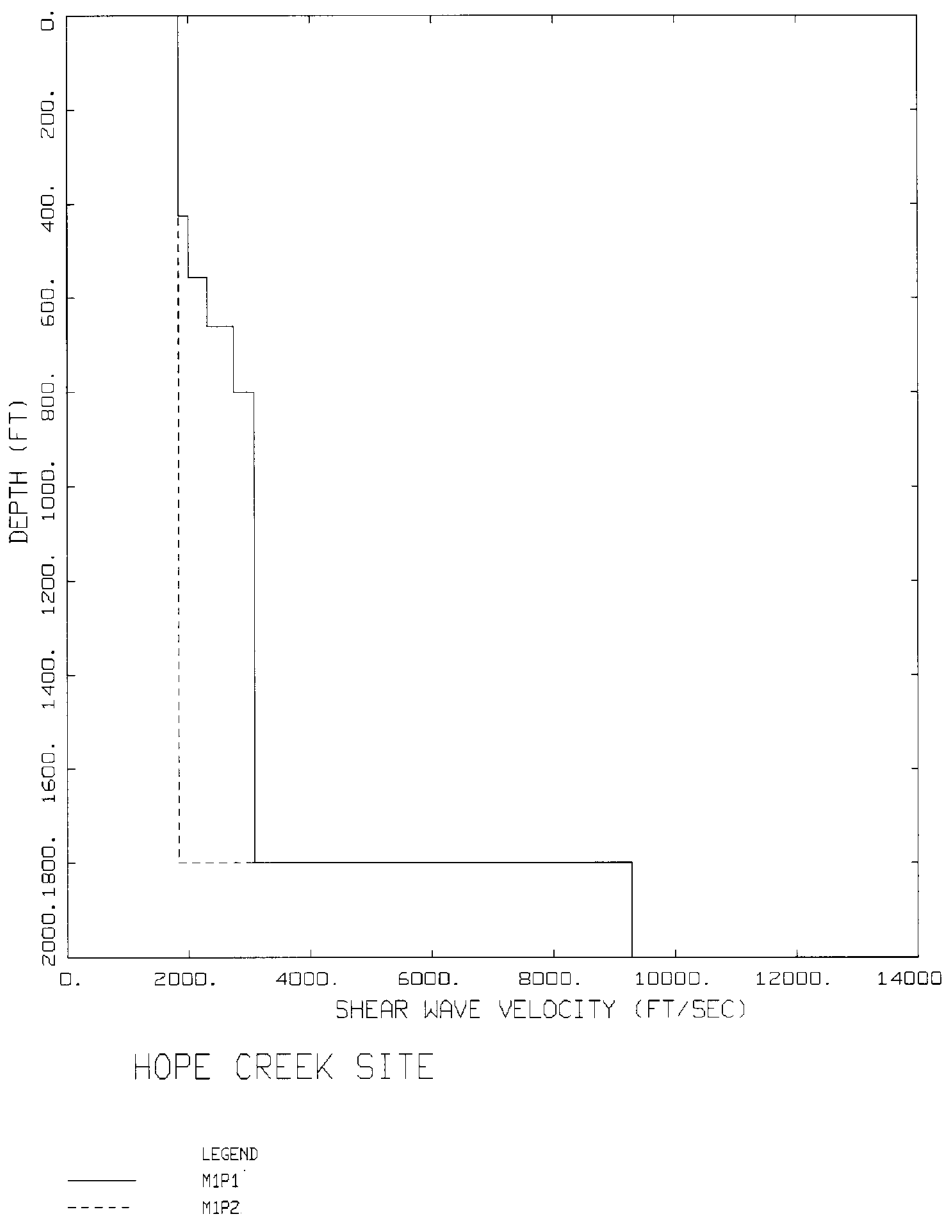

Figure B-6
Shear-Wave Velocity Profiles for the Hope Creek Site

Profile M1P1 is considered the base case profile with M1P2 to accommodate a continued low velocity in the deep sands, clays, and gravels. Both profiles are estimates below a depth of about 400 ft (122m).

B.6.2.2 Modulus Reduction and Hysteretic Damping Curves

Since little recent information was available for site-specific nonlinear dynamic material properties, Peninsular Range modulus reduction and damping curves were taken as base-case properties (Silva et al., 1997; 1998b). These curves are based on modeling recorded strong ground motions in Southern California, are more linear than the EPRI (1993) cures, and are considered to reflect expected non-linearity of the Pleistocene (and earlier) soil (sands and clays) beneath the power block. To consider the potential influence of gravels on the soil non-linearity, the more nonlinear EPRI (1993) curves are considered as well (M2P1).

B.6.2.3 Regional Crustal Damping (kappa)

Based on location within the same physiographic province as the Savannah River Site and similar thickness of soils (about 1,000 ft, 305m for the Savannah River Site), the measured Savannah River kappa value of 0.02 sec (Fletcher, 1995) was adopted as the base-case value. A 50% increase, due to the deeper depth to Precambrian basement of about 1,800 ft (550m), was also considered (M1P1.KH).

B.6.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-6.

Table B-6
Hope Creek Weights

Properties*	Category Weights
M1P1	0.7
M1P1.KH	0.3
P1	0.7
P2	0.3
M1	0.7
M2	0.3
	Combined Weights
M1P1	0.343
M1P1.KH	0.210
M1P2	0.210
M2P1	0.147
M2P2	0.090

*M1P1; base case profile, Peninsular Range Curves, kappa = 0.02 sec

M1P1.KH; base case profile, Peninsular Range Curves, kappa = 0.03 sec

M1P2; low gradient profile, Peninsular Range Curves, kappa = 0.02 sec

M2P2; low gradient profile, EPRI Curves, kappa = 0.02 sec

M2P1; base case profile, EPRI Curves, kappa = 0.02 sec

B.7 LA SALLE SITE

The La Salle County Nuclear Generating Station is located in Northeastern Illinois at the northern end of the Illinois Basin.

The site is underlain by Pleistocene Wisconsinan Wedron silty clay glacial till with some localized sand and gravel deposits. The thickness of the till in the area of the plant structures is about 170 ft (52m). The glacially derived Pleistocene deposits unconformably overlie a complex series of Paleozoic shales, sandstones, siltstones, clays, coals, and limestones with a total thickness of about 4,000 ft (1,220m). Precambrian crystalline basement rock was estimated to be at a depth of about 4,200 ft (1,280m) (EPRI, 1989).

B.7.1 Soil Profile Information

The top 44 ft (13.4m) of the Pleistocene Wedron Formation was removed during construction of the reactor building. The thickness of the remaining till underneath the reactor building is about 126 ft (38.4m). The Paleozoic strata consist of the Pennsylvanian Carbondale (151 ft (46m) thick) and Spoon Formations (25 ft (7.6m) thick). Drilling penetrated both of these formations during the site investigation. The remaining Paleozoic strata include about 600 ft (180m) of interbedded limestones, dolomites, sandstone and shale overlying about 3,300 ft (1,000m) of Cambrian sandstone, shale, and dolomite. Basement rock is estimated to be at a depth of 4,200 ft (1,280m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical refraction studies measured compressional seismic velocities into the top of the Paleozoic shales and siltstones at a depth of 170 ft (52m). The estimated (from Vp and estimate of Poisson’s ratio) shear-wave velocities are 400 ft/sec (122 m/sec), 1,640 ft/sec (500 m/sec) and 4,800 ft/sec (1,463 m/sec) in the upper till, lower Till and Paleozoic Formations, respectively (EPRI, 1989).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site (EPRI, 1989).

B.7.2 Description of Base Case Profiles

B.7.2.1 Shear Wave Velocity Profiles

Figure B-7 shows the base-case profile (M1P1) to a depth of 4,200 ft (1,280m) (randomized + 1,000 ft (305m) where it encounters Precambrian basement. To consider the likelihood of a gradient in the deep sandstones, shales, and dolomites, profile M1P2 (Figure B-7) has shear-wave velocity increasing with depth.

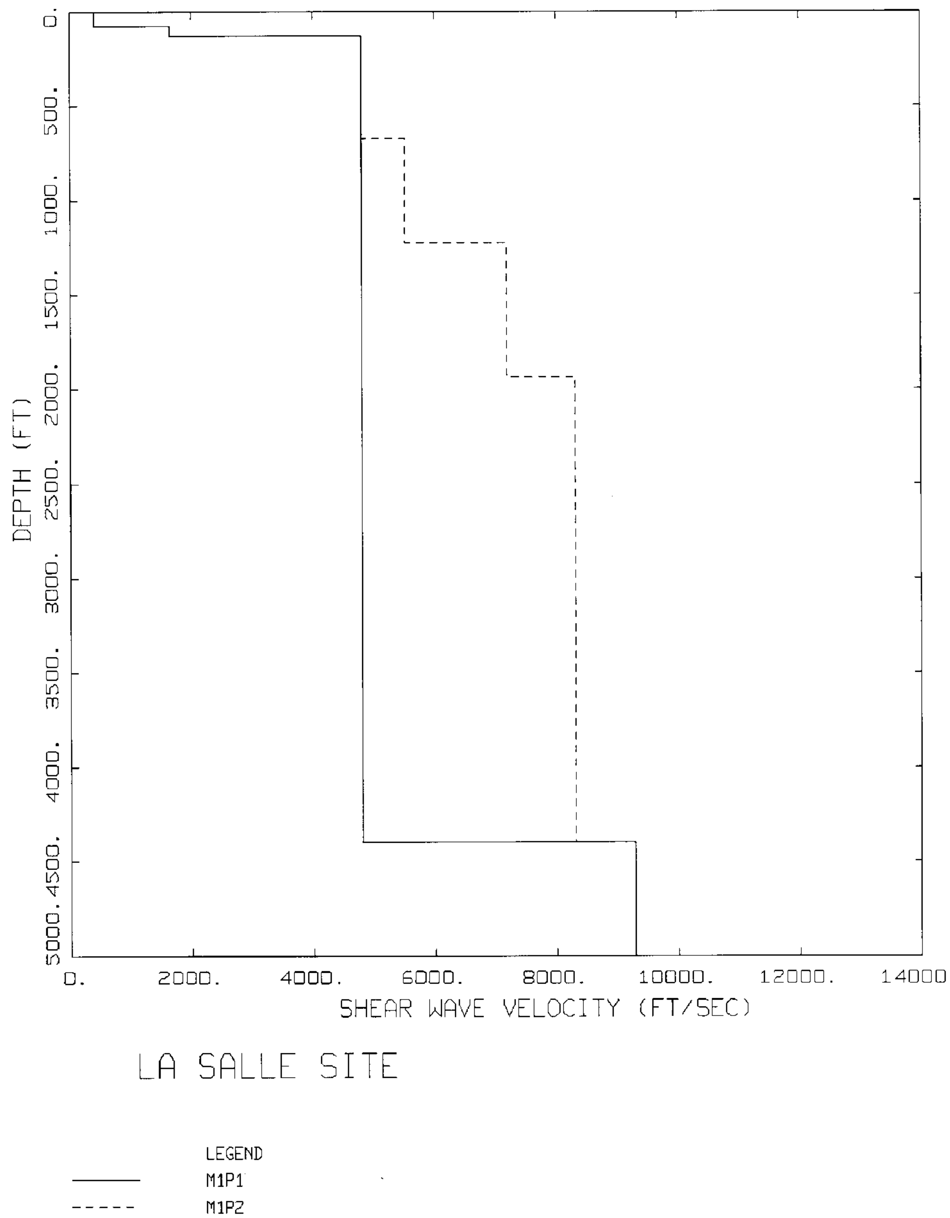

Figure B-7
Shear-Wave Velocity Profiles for the La Salle Site

Profile M1P1 is considered the base case profile with M1P2 to accommodate a gradient in the deep sandstone and dolomite. Both profiles reflect estimates below a depth of about 600 ft (183m).

B.7.2.2 Modulus Reduction and Hysteretic Damping Curves

For the till section of the profile, approximately the top 126 ft (38.4), EPRI Till curves (EPRI, 1993) were used (M1 in Table B-7). To consider the possibility that the upper and lower Till behaves at high strain in a manner similar to typical cohesion-less soil, the EPRI (1993) curves for sands, gravels, and low PI clays were also used (M2 in Table B-7).

B.7.2.3 Regional Crustal Damping (kappa)

Using Equation B-1 and considering the Carbondale shale and siltstone as outcropping with an average shear-wave velocity over the top 100 ft (31m) at 4,800 ft/sec (1,463 m/sec), the estimated kappa value is 0.0157 sec. For the overlying Till section, the low-strain damping in the EPRI (1993) Till curves contribute a kappa of 0.007 sec while the EPRI (1993) soil curves contribute 0.005 sec, resulting in a total site kappa of 0.0223 sec and 0.0207 sec respectively.

To consider alternative mean kappa values, the rock outcrop was considered to have a kappa value of 0.01 sec, resulting in a total site kappa of 0.0166 (M1P1.KL in Table B-7). For a high kappa value, the total kappa was taken as 0.04 sec, an overall conservative average value for western North America rock and soil sites (Silva et al., 1997).

B.7.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-7.

Table B-7
La Salle Weights

Properties*	Category Weights
M1P1	0.4
M1P1.KH	0.2
M1P1.KL	0.4
P1	0.5
P2	0.5
M1	0.5
M2	0.5
	Combined Weights
M1P1	0.1
M1P1.KH	0.1
M1P1.KL	0.2
M1P2	0.5
M2P1	0.1

*M1P1; base case profile, Till Curves, kappa = 0.0223 sec

M1P1.KH; base case profile, Till Curves, kappa = 0.04 sec

M1P1.KL; base case profile, Till Curves, kappa = 0.0166 sec

M1P2; low gradient profile, Till Curves, kappa = 0.0223 sec

M2P1; base case profile, EPRI Curves, kappa = 0.0207 sec

B.8 NINE MILE POINT SITE

The Nine Mile Point Nuclear Station is located on the south shore of Lake Ontario in Oswego County, New York. The site is located in the Erie-Ontario Lowlands physiographic province.

The site is located on 10 to 15 ft (3 to 4.5m) of Pleistocene glacial deposits, a sandy till. Moderately hard Oswego sandstone of Ordovician age lies beneath the till. Thinly bedded silty and clayey lenses are common in the Oswego Formation that is about 175 ft (53m) thick at the site. This formation grades down in to the Lorraine group that consists of shale and siltstone. This group was estimated to be 665 ft (200m) thick. Below these strata are the Ordovician Trenton limestone and Cambrian Potsdam sandstone groups of about 820 ft (250m) and 30 ft (9m) thick, respectively. Crystalline basement rock was estimated to be at a depth of about 1,700 ft (520m) (EPRI, 1989).

B.8.1 Soil Profile Information

The surficial glacial till was removed during construction. The Unit 1 and Unit 2 plant structures are founded on firm bedrock consisting of Oswego sandstone or Lorraine shale, respectively. Basement rock is estimated to be at a depth of 1,700 ft (520m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical measurements including seismic refraction, uphole, and cross-hole were performed from 1964 to 1978. During the cross-hole survey, measurements of shear-wave velocity were made to a depth of 350 ft (106.7m). The measured shear-wave velocity in the Oswego sandstone at depth was about 8,000 ft/sec (2,438 m/sec). Inferred shear-wave velocities based on compression-wave velocities range from about 5,000 ft/sec (1,524 m/sec) to about 8,000 ft/sec (2,438 m/sec) over the shallow portion of the sandstones below the surficial till. A 3D geophysical survey showed a range in shear-wave velocities from 3,600 ft/sec (1,097 m/sec) to about 7,000 ft/sec (2,133 m/sec) (EPRI, 1989).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.8.2 Description of Base Case Profiles

B.8.2.1 Shear Wave Velocity Profiles

Figure B-8 shows the base-case profile (M1P1) with a steep gradient in the shallow sandstone reaching hard rock velocities (9,285 ft/sec, 2.83 km/sec) at a depth of about 60 ft (18m) (randomized + 40 ft, 12m). To accommodate the range in inferred and measured shear-wave velocities, profile M1P2 (Figure B-8) considers hard rock as foundation material and profile M1P3 (Figure B-8) assumes a low near surface velocity of 5,000 ft/sec (1,524 m/sec) extends to basement material at a depth of 1,700 ft (518m) (randomized + 500 ft, 152m).

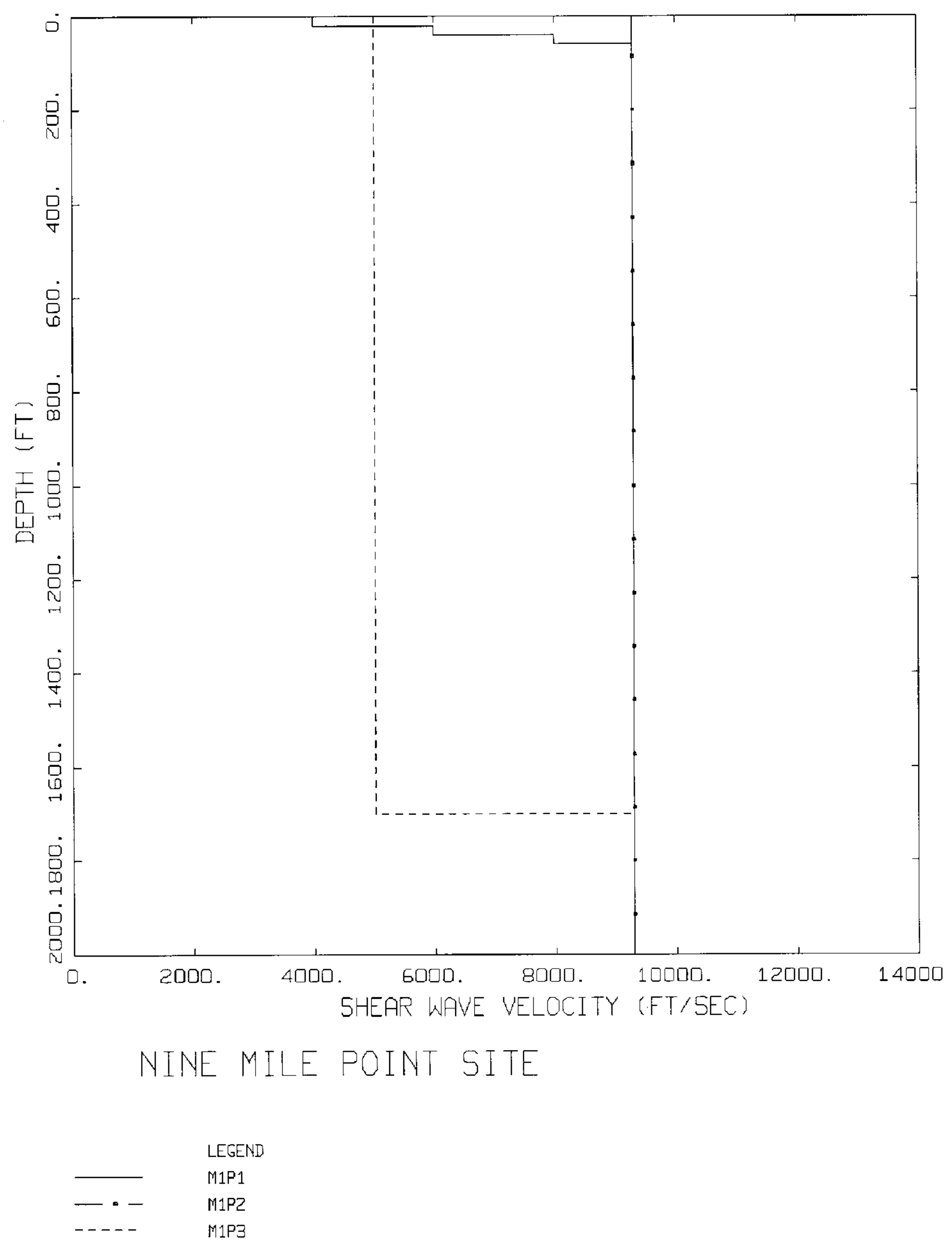

Figure B-8
Shear-Wave Velocity Profiles for the Nine Mile Point Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate higher and lower velocities at the surface and with depth. The range in profiles is intended to capture the range in inferred (from shallow compressional-wave refraction) and measured (crosshole) shear-wave velocities.

B.8.2.2 Modulus Reduction and Hysteretic Damping Curves

For the all three profiles (Figure B-8) the Peninsular Range rock curves (Silva et al., 1997) are used for the very shallow materials (approximately 60 ft, 18m).

B.8.2.3 Regional Crustal Damping (kappa)

For the base-case profile (M1P1, Figure B-8), the average shear-wave velocity over the top 100 ft (31m) is nearly 7,000 ft/sec (2,133 m/sec) resulting in an estimated kappa value of 0.01 sec using Equation B-1. The high velocity profile (M1P2) has the defined central and eastern North America hard rock kappa value of 0.006 sec (amplification of 1.0). For the low-velocity profile, with a shear-wave velocity of 5,000 ft/sec (1,524 m/sec), Equation B-1 gives an expected kappa value of 0.015 sec (M1P3).

To consider alternatives for the base case profile M1P1.KL and M1P1.KH have kappa values of 0.006 and 0.020 sec, respectively.

B.8.2.4 Profile Weights

Profile weighs used for the amplification factors are listed below in Table B-8.

Table B-8
Nine Mile Point Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.2
M1P1.KL	0.2
M1P1	0.5
M1P2	0.3
M1P3	0.2
	Combined Weights
M1P1	0.3
M1P1.KH	0.1
M1P1.KL	0.1
M1P2	0.3
M1P3	0.2

*M1P1; base case profile, kappa = 0.01 sec

M1P1.KH; base case profile, kappa = 0.02 sec

M1P1.KL; base case profile, kappa = 0.006 sec

M1P2; high velocity profile, kappa = 0.006 sec

M1P3; low velocity profile, kappa = 0.015 sec

B.9 NORTH ANNA SITE

The North Anna Power Station is located on the southern shore of Lake Anna in Northeastern Virginia. The site is located in the central part of the Piedmont physiographic province.

The site is located on a Paleozoic basement rocks consisting of granitic gneiss. A thin soil and a zone of weathered bedrock (saprolite) overlie slightly weathered to fresh un-weathered crystalline bedrock, which is encountered at depths of about 40 ft (12.2m) across the plant area (EPRI, 1989).

B.9.1 Soil Profile Information

The reactor buildings are founded crystalline bedrock at a depth of about 68 ft (20.7m) below finished grade. Basement rock is estimated to be at a depth of 100 ft (30.5m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical measurements included seismic refraction, in-hole (Birdwell 3D logs), and cross-hole were performed (EPRI, 1989). For the deepest hole, shear-wave velocities around 8,000 ft/sec (1,838 m/sec) are measured at depths of 130 ft (40m). The range in measured shear-wave velocities was from about 4,000 ft/sec (1,219 m/sec) to about 8,000 ft/sec (1,838 m/sec). Dominion (2003) completed a downhole seismic test that determined shear-wave velocities to a depth of 67.5 ft (20.6m) for the North Anna site. The velocity at this depth was 6,030 ft/sec (1,838 m/sec).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.9.2 Description of Base Case Profiles

B.9.2.1 Shear Wave Velocity Profiles

Figure B-9 shows the base-case profile (M1P1). The surface shear-wave velocity is 6,030 ft/sec (1,838 m/sec) and is based on a recent suspension log survey with this velocity encountered at a depth of about 60 ft (18m), the reactor building foundation depth. The increase in velocities below reflects application of a gradient taken from crosshole seismic tests in similar Piedmont physiographic province materials (Catawba Site, Section B-3). To accommodate higher outcrop velocities, profile M1P2 has hard rock outcropping at the surface (reactor containment depth) with a shear-wave velocity of 9,285 ft/sec (2.83 km/sec). To consider a low surficial velocity and high velocity gradient, profile M1P3 (Figure B-9) is based on the low range of crosshole seismic tests (EPRI, 1989). Profile M1P1, the base-case, is assumed to encounter hard rock conditions at a depth of 119 ft (36m) (randomized + 33 ft (10m)). For the high gradient profile (M1P3) hard rock is at a depth of 139 ft (42m), randomized + 50 ft (15m).

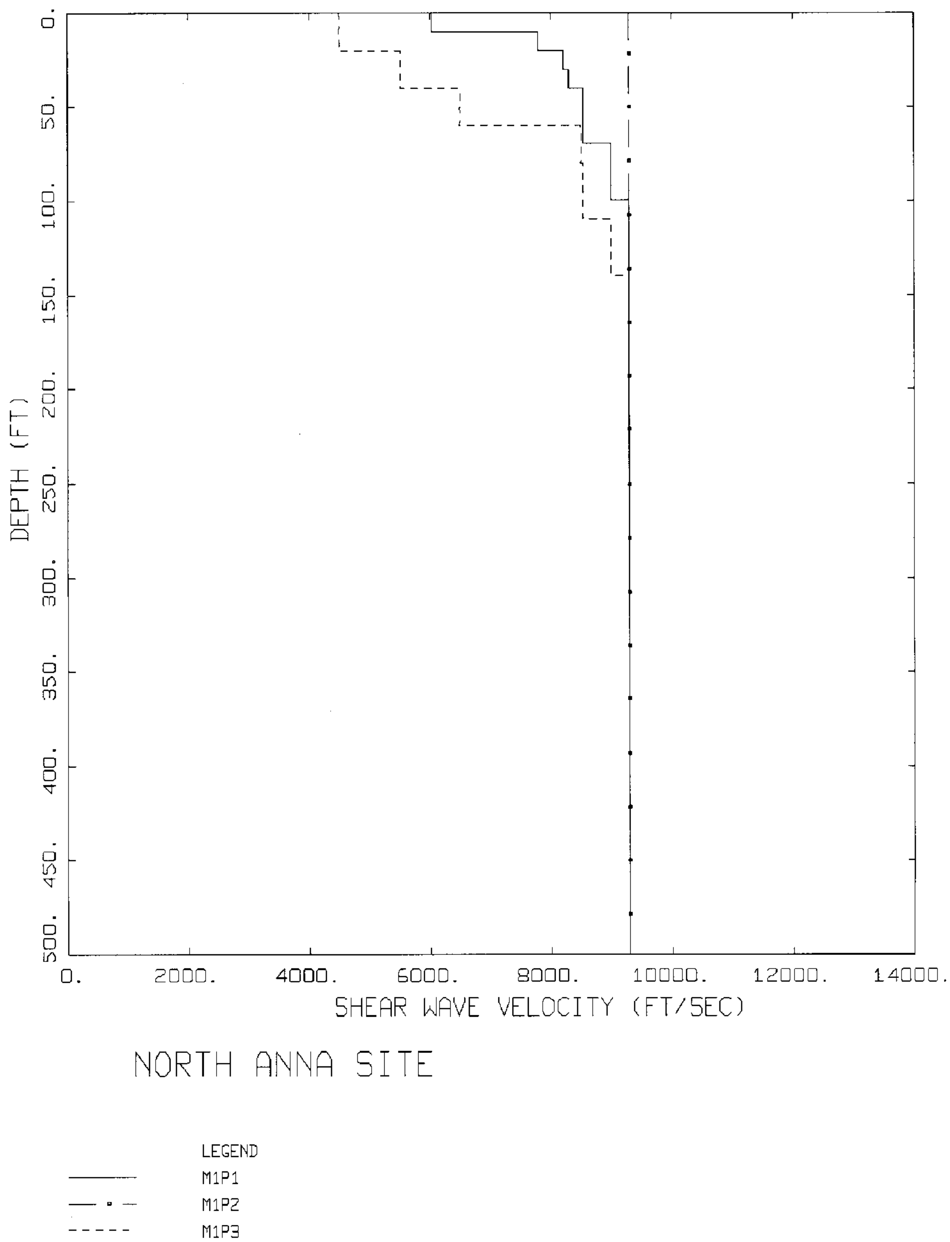

Figure B-9
Shear-Wave Velocity Profiles for the North Anna Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate higher and lower velocities. All three profiles are estimates below a depth of about 50 ft (15m).

B.9.2.2 Modulus Reduction and Hysteretic Damping Curves

In the shallow potion of the profiles, approximately top 100 ft (31m) the Peninsular Range rock curves (Silva et al., 1997) are used.

B.9.2.3 Regional Crustal Damping (kappa)

The kappa value for the base-case profile was taken as the hard rock value of 0.006 sec plus that contributed by the low-strain damping from the Peninsular Range rock curves (Silva et al., 1997), a value of 0.0008 sec, for a total kappa of about 0.007 sec. With an average shear-wave velocity over the top 100 ft (31m) of just over 8,000 ft/sec (2,438 m/sec), Equation B-1 gives a similar value of 0.0087 sec. High velocity profile M1P2 (Figure B-9) has the reference kappa of 0.006 sec (amplification of 1.0) while the low velocity (high gradient) profile (M1P3) was given a total kappa value of 0.012 sec, based on Equation B-1 with an average shear-wave velocity (100 ft, 31m) of 6,384 ft/sec (1946 m/sec).

B.9.2.4 Profile Weights

The B-9 lists profile weights used for the amplification factors.

Table B-9
North Anna Weights

Properties*	Category Weights
M1P1	0.5
M1P2	0.1
M1P3	0.4
	Combined Weights
M1P1	0.5
M1P2	0.1
M1P3	0.4

*M1P1; base case profile, kappa = 0.007 sec

M1P2; high velocity profile, kappa = 0.006 sec

M1P3; low gradient profile, kappa = 0.012 sec

B.10 RIVER BEND SITE

The River Bend Station about 24 miles (39 km) northwest of Baton Rouge, Louisiana on the Uplands complex adjacent to the Mississippi alluvial valley. The site is in the Southern Hills physiographic section of the Gulf Coastal Plain physiographic province. The plant area is situated 1.9 mi (3.3 km) northeast of the east bank of the Mississippi River adjacent to the Deltaic physiographic province. In the site vicinity the Uplands are composed of Plio-Pleistocene fluvial deposits with an overlying blanket of loess.

The near surface stratigraphy is consists of about 8 ft (2.4m) of loess over the Pleistocene Port Hickey Top Stratum and terrace deposits 60 ft (18m) thick. Beneath these strata are silty sands, sands, clays, and gravels of the Pliocene Citronelle Formation and the hard clay of the Pascagoula Formation. The Pascagoula Formation was the oldest formation encountered by borings in the site area. It is a part of the Grand Gulf – Fleming Group that is about 6,500 ft (2,000m) thick at the site. The strata underlying the site consist of a thick and stratigraphically complex sequence of relatively flat lying sediments that are part of the Gulf Coast geosyncline. These sediments are about 20,000 ft (6,000m) thick and unconformably overlie a sequence or rocks composed mainly of Mesozoic limestone. Precambrian crystalline basement rock was estimated to be at a depth of about 27,000 ft (8,200m) (EPRI, 1989).

B.10.1 Soil Profile Information

The Loess, Port Hickey, and top 20 ft (6m) of the Citronelle deposits were removed at the site to a depth of 88 ft (27m). The reactor building is founded on 40 ft (12m) of compacted fill on top 60 ft (18m) of fine to medium sand and gravel (Citronelle Buried Channel Deposits). Underlying the Citronelle is several thousand feet of hard clay (Pascagoula Formation). Rock hazard defined as basement material with a Vs of 2.83 km/sec is taken at a depth of 5,000 ft (1,524m) which is deep enough to accommodate soil amplification at the lowest frequency of interest, 0.5 Hz (Silva et al., 1999, 2000).

Geophysical measurements included seismic refraction, downhole, uphole and cross-hole were performed. In the deepest boring shear-wave velocities around 1,200 ft/sec (365 m/sec) are measured at depths of 210 ft (64m) in the Pascagoula clay. Shear-wave velocity for the compacted fill is calculated from estimates of shear moduli and density at about 700 ft/sec (213 m/sec).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.10.2 Description of Base Case Profiles

B.10.2.1 Shear Wave Velocity Profiles

Shown in Figure B-10 are the base-case (M1P1), low deep velocity (M1P2), and high deep gradient (M1P3) profile with all three reflecting measured shear-wave velocities in the top roughly 120 ft (37m) (top 90 ft (27m) removed for embedment of reactor building). The top 40 ft (12m) reflecting compacted fill underlying the reactor building. Below the deepest measured velocity (1,200 ft/sec) (365 m/sec) the base-case velocities were assumed to increase to about 2,000 ft/sec (610 m/sec) at a depth of about 500 ft (152m), based on profiles in the Uplands province in the northern portion of the embayment (Entergy, 2003). The low-velocity profile (M1P2, Figure B-10) reflects the assumption of a continued Uplands 500 ft (112m) depth velocity to a depth of 3,281 ft (1 km), taken as a fictitious depth to basement to allow amplification to the lowest frequency of interest, 0.5 Hz. The high gradient profile (M1P3) accommodates the possibility of a rapidly increasing velocity with depth, reaching firm rock conditions around 1,000 ft to 2,000 ft (305m to 610m). All three profiles had depth to basement material at 3,281 ft (1 km) and randomized ± 500 ft (152m).

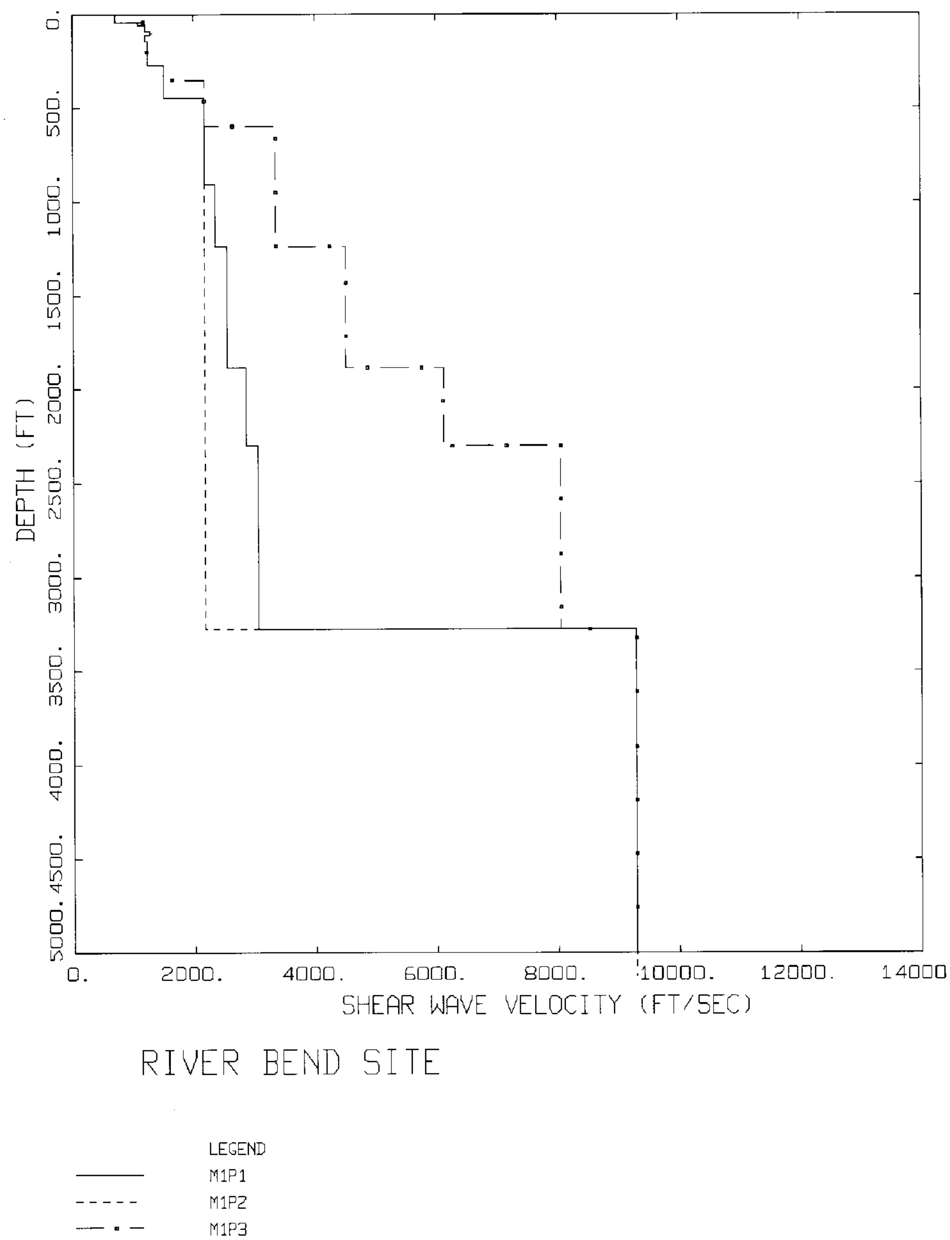

Figure B-10
Shear-Wave Velocity Profiles for the River Bend Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate lower and higher velocities at depth. Profiles reflect estimates below measured velocities depth of about 100 ft (31m).

B.10.2.2 Modulus Reduction and Hysteretic Damping Curves

For the compacted fill and Pleistocene terrace deposits of the Citronelle sands, clays, and gravels over the top approximately 98 ft (30m), the EPRI (1993) 250 ft to 500 ft (76m to 152m) were used. For the Pascagoula clays, index properties suggested a PI of about 20% (EPRI, 1989) and resonant column tests showed modulus reduction curves consistent with Vucetic and Dobry (1991) cohesive soil curves reflecting a PI closer to 50% (EPRI, 1989). As a result the Vucetic and Dobry (1991) curves for a PI of 50% were used for depths between 98 ft (30m) and 272 ft (83m). For the clays below this depth the Vucetic and Dobry curves for a PI of 100% were used, to accommodate the potential effects of confining pressure. Below a depth of about 450 ft (137m), the EPRI (1993) 500 ft to 1,000 ft (152m to 304m) curves were used, consistent with the deep clays at the Savannah River Site (SRS, 1996).

B.10.2.3 Regional Crustal Damping (kappa)

As with the Grand Gulf Site (Section B.5), located in the Uplands complex of the Mississippi embayment, the base-case total site kappa value was taken as 0.046 sec. Higher and lower kappa values based on a 50% variation of the base-case value giving 0.069 sec (M1P1.KH in Table B-10) and 0.031 sec (M1P1.KL in Table B-10) were used as well.

B.10.2.4 Profile Weights

Table B-10 lists the weights used for the amplification factors.

Table B-10
River Bend Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.2
M1P1.KL	0.2
M1P1	0.8
M1P2	0.1
M1P3	0.1
	Combined Weights
M1P1	0.48
M1P1.KH	0.16
M1P1.KL	0.16
M1P2	0.10
M1P3	0.10

*M1P1; base case profile, kappa = 0.046 sec

M1P1.KH; base case profile, kappa = 0.069 sec

M1P1.KL; base case profile, kappa = 0.031 sec

M1P2; high velocity profile, kappa = 0.046 sec

M1P3; low velocity profile, kappa = 0.046 sec

B.11 SHEARON HARRIS SITE

The Shearon Harris Nuclear Power Plant is located near the northern end of a reservoir on Buckhorn Creek in the extreme southwest corner of Wake County and the southeast corner of Chatman County in North Carolina. The site is located in the Triassic belt subdivision of the Piedmont Plateau physiographic province, a deeply eroded plateau-like segment of the Appalachian Mountain System. The site is located in the south central part of the Durham Basin that is about 52 miles (84 km) long with a maximum width of 20 miles (43 km) which is the northern most of three basins within the Deep River Triassic Basin.

The main plant structures are located on Triassic-age Sanford Formation consisting of gently dipping, well-consolidated sandstone, siltstone, and shaly siltstone. A thin residual soil and a zone of weathered bedrock overlie dense, massive sedimentary bedrock, which is encountered at depths of 16 ft (4.9m) across the plant area. Crystalline basement rock was estimated to be at a depth of at least 6,000 ft (1,829m) (EPRI, 1989).

B.11.1 Soil Profile Information

The reactor buildings are founded on siltstone and sandstone bedrock at an embedment depth of about 26 ft to 81 ft (8m to 25m) below finished grade. The bedrock at the site was originally overlain by about 8 ft (2.4m) of residual soils and about 8 ft (2.4m) of weathered and fractured rock. Basement rock is estimated to be at a depth over 6,000 ft (1,829m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical measurements included seismic refraction, shear-wave velocity, and up-hole compressional-wave velocity measurements. Shear-wave velocity measurements at the top of the Sanford Formation (reactor buildings foundation material) are 5,600 ft/sec (1,707 m/sec) (EPRI, 1989).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.11.2 Description of Base Case Profiles

B.11.2.1 Shear Wave Velocity Profiles

Figure B-11 shows the base-case profile (M1P1) with an embedment depth shear-wave velocity for the sandstones and siltstones of 5,600 ft/sec (1,707 m/sec) extending to crystalline basement, taken at a depth 6,000 ft (1,829m) and randomized $\pm$ 2,000 ft (610m). To consider the effects of an increase in velocities with depth, profile M1P2 (Figure B-11) considers reaching hard rock conditions, with a shear-wave velocity of 9,285 ft/sec (2.83 km/sec), at a depth of 500 ft (152m) (randomized $\pm$ 300 ft, 91m).

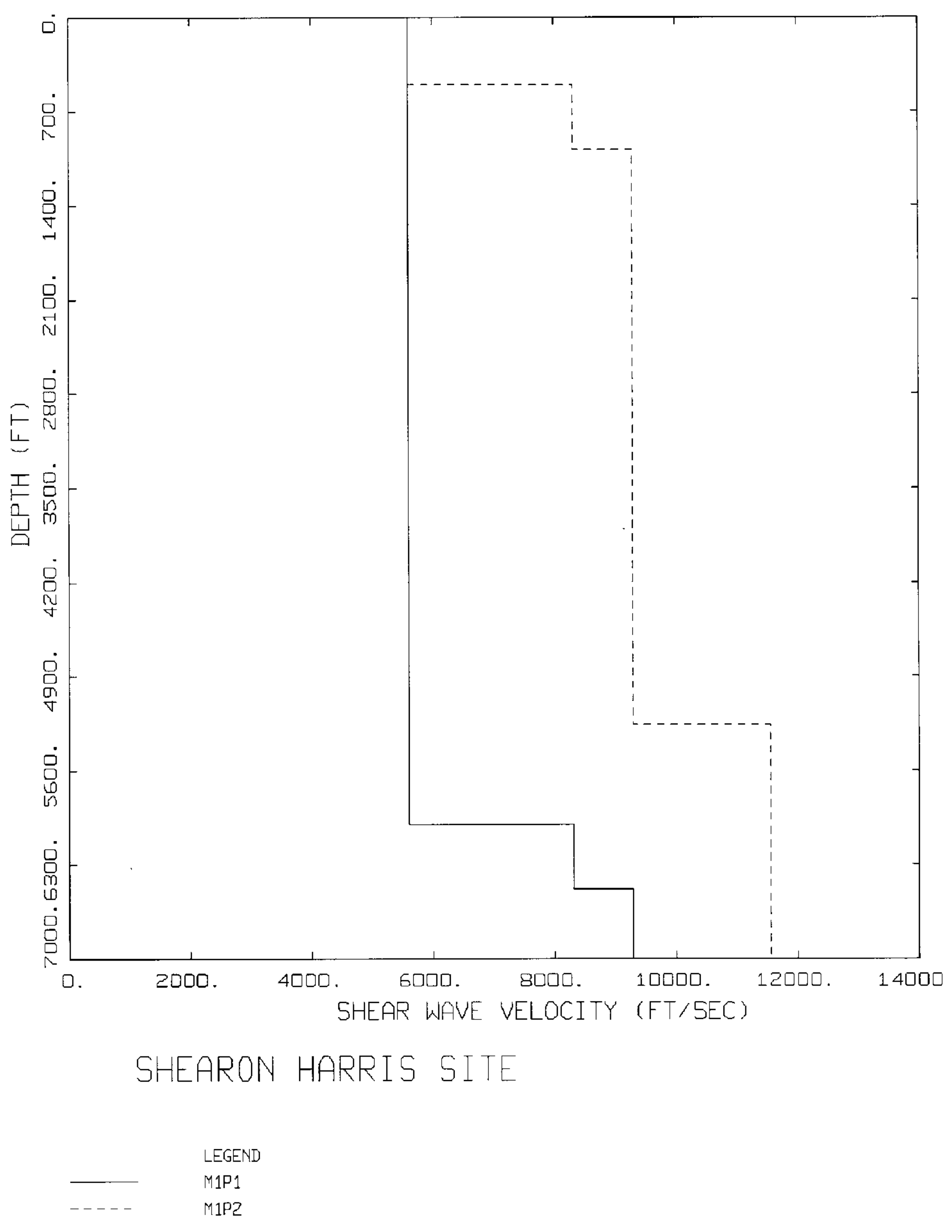

Figure B-11
Shear-Wave Velocity Profiles for the Shearon Harris Site

Profile M1P1 is considered the base case profile with M1P2 to accommodate a gradient in the deep sandstone and dolomite. Both profiles reflect estimates below a depth of about 50 ft (15m).

B.11.2.2 Modulus Reduction and Hysteretic Damping Curves

To accommodate possible nonlinear effects in the shallow portions of the profiles, the Peninsular Range rock curves (Silva et al., 1997) were used over the top 100 ft (31m).

B.11.2.3 Regional Crustal Damping (kappa)

For the outcropping sandstones and siltstones, the average shear-wave velocity over the top 100 ft (31m) is 5,600 ft/sec (1,707 m/sec) resulting in a kappa value of 0.0132 sec (Equation B-1). High and low kappa values considering a ± 50% variation in the base-case value were also considered. These values were 0.0198 sec (M1P1.KH in Table B-11) and 0.0088 sec (M1P1.KL in Table B-11) respectively.

B.11.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-11.

Table B-11
Shearon Harris Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.2
M1P1.KL	0.2
M1P1	0.7
M1P2	0.3
	Combined Weights
M1P1	0.42
M1P1.KH	0.14
M1P1.KL	0.14
M1P2	0.30

*M1P1; base case profile, kappa = 0.0132 sec

M1P1.KH; base case profile, kappa = 0.0198sec

M1P1.KL; base case profile, kappa = 0.0088 sec

M1P2; high gradient profile, kappa = 0.0132 sec

B.12 SOUTH TEXAS SITE

The South Texas Project is located along the west bank of the Colorado River about 15 miles (24 km) from the Gulf of Mexico near Bay City, Texas. The site is in the Texas Gulf Plain physiographic province that is dominated by a thick sedimentary prism known as the Gulf Coast Geosyncline.

The near surface stratigraphy consists of about 700 ft to 800 ft (200m to 250m) of Pleistocene Beaumont and Lissie Formations. The upper 300 ft (90m) consists of layers of silty sand and clay with some sandy silt and fine sand. Quaternary sediments are present to at least 2620 ft (800m) beneath the site. The base of the Miocene Oakville sandstone is at about 6,200 ft

(1,900m). Pre-Cretaceous basement rock was estimated to be at a depth of about 34,500 ft (10,500m) (EPRI, 1989).

The strata above the pre-Cretaceous basement rock include 26,000 ft (7,900m) of Cenozoic sediments underlain by 8,500 ft (2,600m) of Cretaceous rocks. Basement rocks consists of 17,000 ft (5,200m) of pre-Cretaceous units that rest on rocks with high seismic velocities that have been termed “lower continental or oceanic crust” (EPRI, 1989).

B.12.1 Soil Profile Information

The top 60 ft (18m) of soil deposits (Layers A, B, C and D; EPRI, 1989) were removed at the site for construction of the reactor containment building. The reactor building foundations are supported by a dense to very dense, slightly silty sand of the Beaumont Formation. The upper 300 ft (91m) of soil generally consists of alternating layers of stiff to hard silty clay and dense to very dense silty sand. Hard rock hazard is defined at basement material with a Vs of 2.83 km/sec.

Shear-wave velocities were measured during cross-hole tests. The reactor building foundations are supported by sand with a shear-wave velocity of 1,150 ft/sec (350 m/sec). In the deepest boring shear-wave velocities around 1,585 ft/sec (483 m/sec) are measured at depths of 341 ft (104m) in the Pleistocene soils.

Dynamic testing of representative soils samples were performed on the natural soils and compacted backfill to estimate modulus reduction and damping curves EPRI (1989). Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.12.2 Description of Base Case Profiles

B.12.2.1 Shear Wave Velocity Profiles

Figure B-12 shows the base-case shear-wave velocity profile (M1P1) based on crosshole measurements over approximately the top 250 ft (76m). Below that depth the Mississippi embayment lowlands profile, which had similar velocities at this depth, was used to extrapolate the base-case profile to a depth of 2,500 ft (762m), to capture potential low-frequency (0.5 Hz) amplification. This generic profile was developed for ground shaking studies in the embayment by Professor Glenn Rix of the MAE Center (personal communication, 2002). The profile is based on a large number of shallow and several deep velocity surveys and extends to a depth of 3,600 ft (1,100m). To accommodate amplification from lower shear-wave velocities beneath 250 ft (76m) (M1P2 in Figure B-12), the deepest measured velocity (1,585 ft/sec (483 m/sec)) was extended to a depth of 1,000 ft (305m), where it was merged to the base-case profile. To consider a steeper velocity gradient, the EPRI (1993) 1,000 ft (305m) stiff sand profile was added to the base of the measured velocities and increased, with a similar gradient, to a depth of 2,500 ft (762m). All three profiles are randomized in depth to 9,285 ft/sec (2.83 km/sec) velocities occurring at a depth of 2,500 ft (726m) and randomized $\pm$ 500 ft (152m).

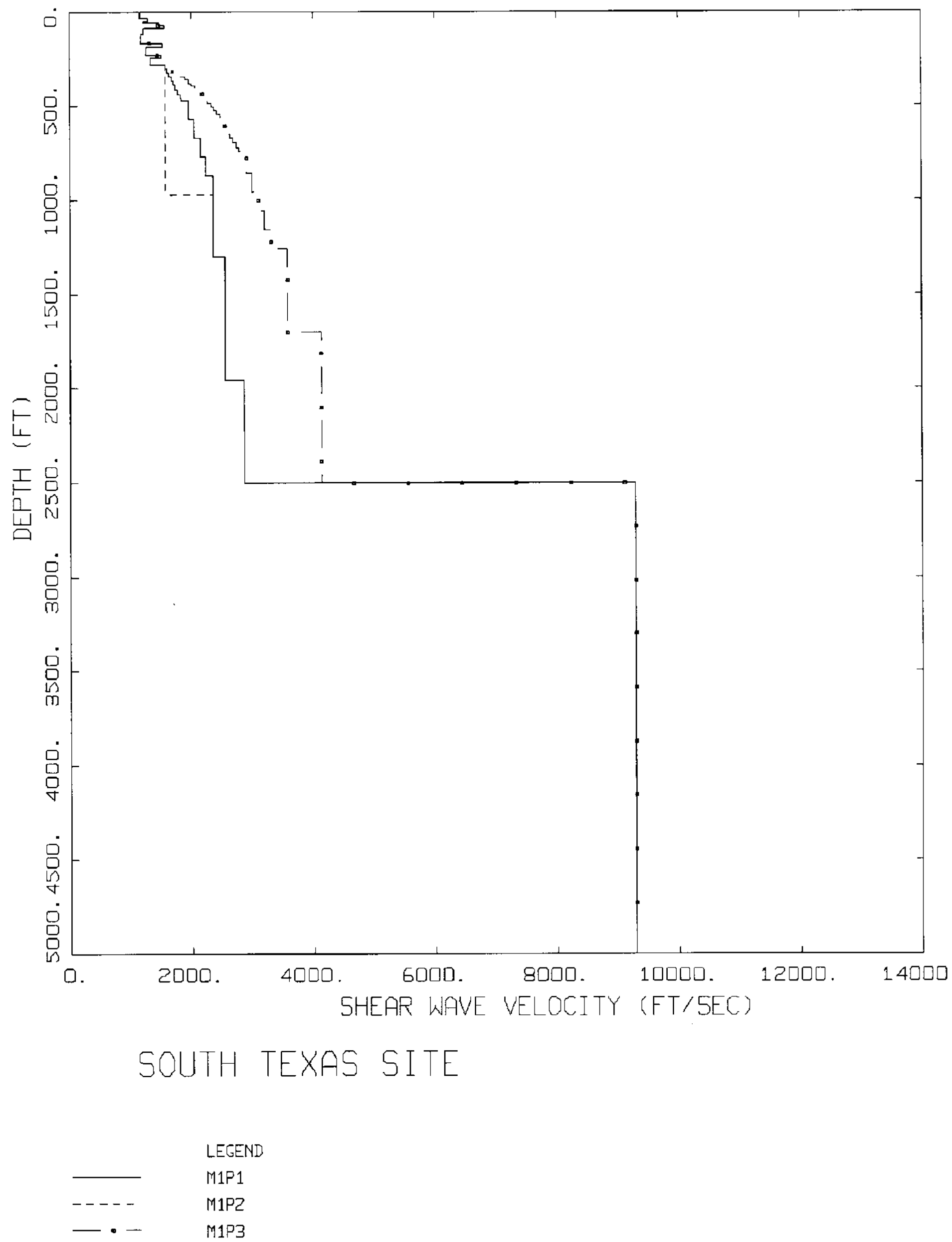

Figure B-12
Shear-Wave Velocity Profiles for the South Texas Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate lower and higher velocities at depth. Profiles reflect estimates below measured velocities depth of about 250 ft (76m).

B.12.2.2 Modulus Reduction and Hysteretic Damping Curves

For the Holocene cohesion-less soils, laboratory dynamic material property testing showed G/G_{max} curves similar to those of EPRI (1993). As a result the EPRI (1993) curves were selected to reflect the base-case dynamic material (M1P1) properties as they are based on more recent testing procedures and have been extensively validated by modeling recorded strong ground motions (Silva et al., 1997; 1998b).

To consider the possibility of more linear response, the Peninsular Range curves (M2P1) were considered as well.

B.12.2.3 Regional Crustal Damping (kappa)

The base-case kappa value was taken as 0.046 sec, as with the other sites located on the deep soils of the Mississippi embayment. High and low kappa values based on a $\pm$ 50% variation about the base-case value were also considered. These alternative values were 0.069 sec and 0.031 sec respectively.

B.12.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-12.

Table B-12
South Texas Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.2
M1P1.KL	0.2
M1P1	0.8
M1P2	0.1
M1P3	0.1
M1	0.6
M2	0.4
	Combined Weights
M1P1	0.288
M1P1.KH	0.160
M1P1.KL	0.160
M1P2	0.100
M1P3	0.100
M2P1	0.192

*M1P1; base case profile, EPRI curves, kappa = 0.046 sec

M1P1.KH; base case profile, EPRI curves, kappa = 0.069sec

M1P1.KL; base case profile, EPRI curves, kappa = 0.031 sec

M1P2; low gradient profile, EPRI curves, kappa = 0.046 sec

M1P3; high gradient profile, EPRI curves, kappa = 0.046 sec

M2P1; base case profile, Peninsular Range curves, kappa = 0.046 sec

B.13 SUMMER SITE

The Virgil C. Summer Nuclear Station is located approximately 1 mile (1.6 km) east of Broad River in Fairfield County in South Carolina. The site is located within the Piedmont physiographic province

The site is located on Paleozoic basement rocks of the Carolina Slate Belt consisting of granodiorite and migmatite. An 80 ft (24m) thick layer of residual soil (saprolite) and a 10 ft (3m) thick zone of weathered and jointed bedrock overlie sound crystalline bedrock, which is encountered at depths of 90 ft (27.4m) across the plant area EPRI (1989).

B.13.1 Soil Profile Information

The reactor buildings are founded on fill concrete overlying weathered rock overlying crystalline bedrock at a depth of about 39 ft (11.9m) below finished grade. Basement rock is estimated to be at a depth of 129 ft (39m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Geophysical measurements included seismic refraction and surface wave testing. Shear-wave velocities for the weathered rock ranged from 1,500 ft/sec to 2,300 ft/sec (460 ft/sec to 700 m/sec) and in the basement rock from 7,400 to 8,000 ft/sec (2,250 to 2,440 m/sec) (EPRI, 1989).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.13.2 Description of Base Case Profiles

B.13.2.1 Shear Wave Velocity Profiles

The base-case profile (M1P1), shown in Figure B-13 has relatively low shear-wave velocities at the surface, about 2,000 ft/sec (610 m/sec). This profile, along with an alternative shallow velocity gradient, having a surface shear-wave velocity of 4,000 ft/sec (1,219 m/sec) (M1P2), are intended to capture portions of the foundations not excavated into firm rock. The criteria used for excavation was a minimum compressional-wave velocity of 8,000 ft/sec (2,438 km/sec) (EPRI, 1989) which, for Poisson's ratios in the 0.35 to 0.4 range in weathered rock, would likely result in a near-surface low shear-wave velocity. Both of the gradient models in Figure B-13 were taken from crosshole measurements at the Catawba site (Section B.3), taken above the reactor containment embedment depth. Both sites are located within the Piedmont physiographic province typified by residual soil (saprolite) overlying weathered and joined bedrock which grades into firm to hard basement material. To consider reactor buildings founded on hard rock, profile M1P3 in Figure B-13 treats the hard rock shear-wave velocity (9,285 ft/sec (2.83 km/sec)) as outcropping at the free surface (embedment depth). Basement depth for profile M1P1 is randomized at 129 $\pm$ 50 ft (39.3 $\pm$ 15.2m). Basement depth for profile M1P2 is randomized at 119 $\pm$ 50 ft (36.3 $\pm$ 15.2m).

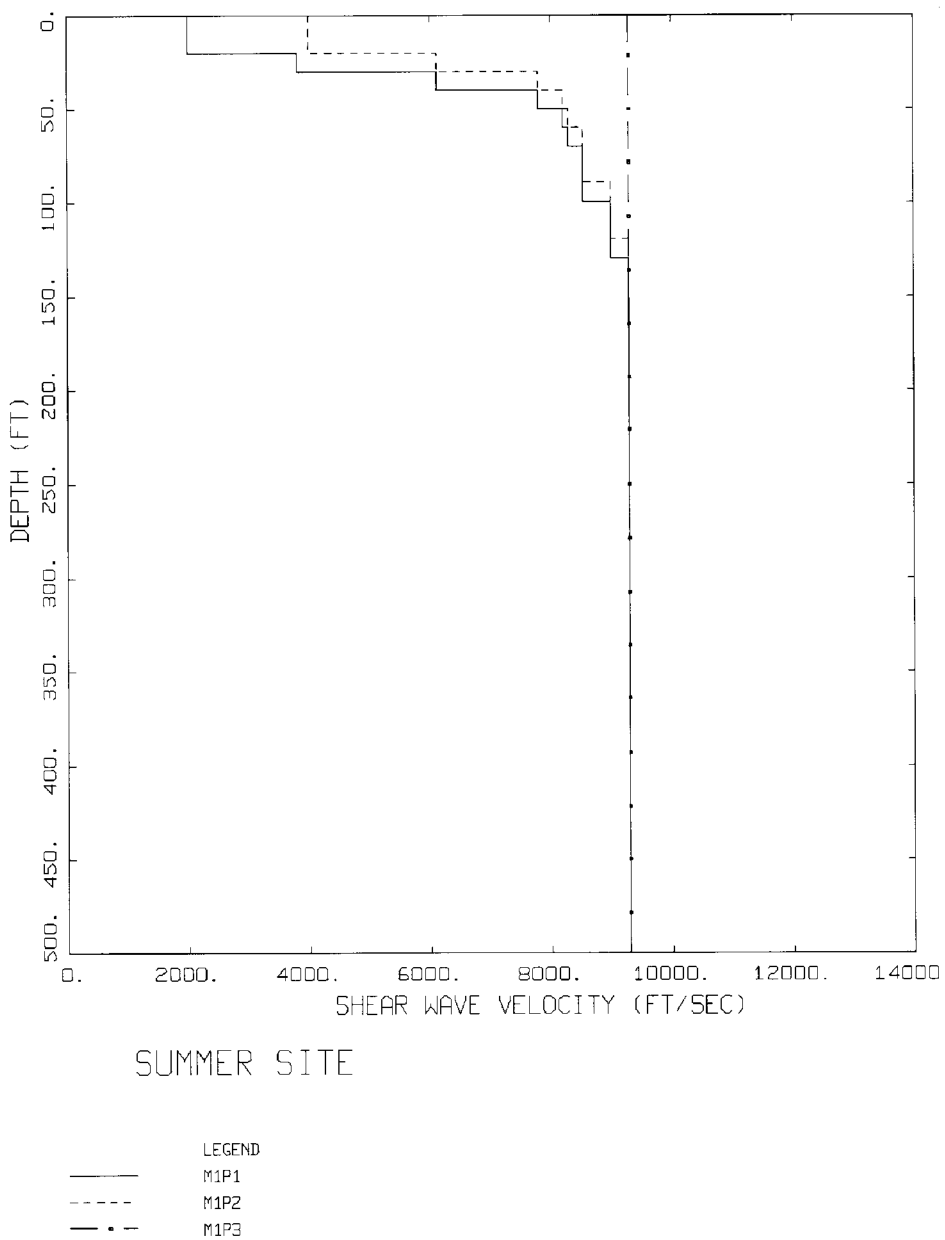

Figure B-13
Shear-Wave Velocity Profiles for the Summer Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate higher near surface velocities. All three profiles are estimates below a depth of about 10 ft (3m).

B.13.2.2 Modulus Reduction and Hysteretic Damping Curves

For the softer profiles, M1P1 and M1P2 in Figure B-13, the Peninsular Range cohesion-less soil G/G_{max} and hysteretic damping curves (Silva et al., 1997) are used to a depth of 20 ft (6m) with the Peninsular Range rock curved to a depth of about 100 ft (31m). Linear response is assumed at greater depths.

B.13.2.3 Regional Crustal Damping (kappa)

For both gradient profiles, M1P1 and M1P2 in Figure B-13, the kappa contributed by the low-strain damping in the nonlinear portions of the profiles (approximately 100 ft (31m)) is 0.001 sec. Adding the hard rock value of 0.006 sec results in a total site kappa value of 0.007 sec. For the hard rock outcropping, the hard rock kappa value of 0.006 sec was used (amplification of 1).

B.13.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-13.

Table B-13
Summer Weights

Properties*	Category Weights
M1P1	0.3
M1P2	0.3
M1P3	0.4
	Combined Weights
M1P1	0.3
M1P2	0.3
M1P3	0.4

*M1P1; base case profile, kappa = 0.007 sec

M1P2; high gradient profile, kappa = 0.007 sec

M1P3; high velocity profile, kappa = 0.006 sec

B.14 THREE MILE ISLAND SITE

The Three Mile Island Nuclear Station is located on Three Mile Island in the Susquehanna River in Dauphin County, Pennsylvania. The site is located in the Gettysburg Basin section of the

Piedmont physiographic province. Three Mile Island was formed as a result of fluvial deposition by the Susquehanna River.

The main plant structures are located on Triassic-age Gettysburg Shale Formation consisting of sandstone, siltstone, and claystone. Surficial materials consist of loose to medium fine silty sand and gravel overlying a layer of medium dense to very dense coarse sand and gravel with numerous boulders and cobbles. Soil depths are 20 ft (6m) in the plant vicinity. There is about 1 to 3 ft (0.3 to 0.9m) of weathered shale beneath the soils. Underlying the Gettysburg Shale is a sequence of lower Paleozoic clastic and carbonate deposits. These strata overlie Precambrian crystalline basement rock estimated to be at a depth of about 16,000 ft (4,800m) (EPRI, 1989).

B.14.1 Soil Profile Information

The reactor buildings are founded on medium hard to hard shale, sandstone, and siltstone bedrock at an embedment depth of about 31 (9.4m) below finished grade.   The sound bedrock at the site was originally overlain by about 20 ft (6.2m) of fluvial deposits. Basement rock is at an approximate depth of 16,000 ft (4,800m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

The compressional-wave velocities at the site were measured by a seismic refraction survey. Compressional-wave velocity measurements at the top of the Gettysburg Shale range from 8,500 ft/sec to 11,000 ft/sec (2,600 m/sec to 3,350 m/sec) (EPRI, 1989). Shear-wave velocities were calculated from compressional-wave velocities with the assumption of Poisson's ratio.

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.14.2 Description of Base Case Profiles

B.14.2.1 Shear Wave Velocity Profiles

Based on a shallow compressional-wave refraction survey giving velocities about 10,000 ft/sec (3,048 m/sec) and assuming a range in Poisson's ratio of 0.3 to 0.4, shear-wave velocities are in the range of 4,000 ft/sec (1,219 m/sec) to 5,000 ft/sec (1,524 m/sec). To accommodate these variabilities, a shear-wave velocity of 4,000 ft/sec (1,219 m/sec) was assumed for the surface and to a depth of 100 ft (31m) where the velocity was increased to 6,000 ft/sec (1,229 m/sec). This base-case profile (M1P1) is shown in Figure B-14. At a depth of about 2,000 ft (610m), the sedimentary rocks, sandstone, shale, and siltstone are taken to reflect an increase in velocity with depth, reaching hard rock velocities (9,285 ft/sec (2.83 km/sec)) at a depth of 5,000 ft (1,524m), deep enough to capture amplification to a frequency of 0.5 Hz. This depth is randomized ± 2,000 ft (610m) to smooth potential resonances. Profile M1P2, Figure B-14 assumes a constant sedimentary rock velocity with depth (to 5,000 ft ± 2,000 ft (1,524m ± 610m)). Profile M1P3 considers the case of encountering hard rock velocity at a depth of about 2,000 ft (610m), randomized ± 500 ft (152m).

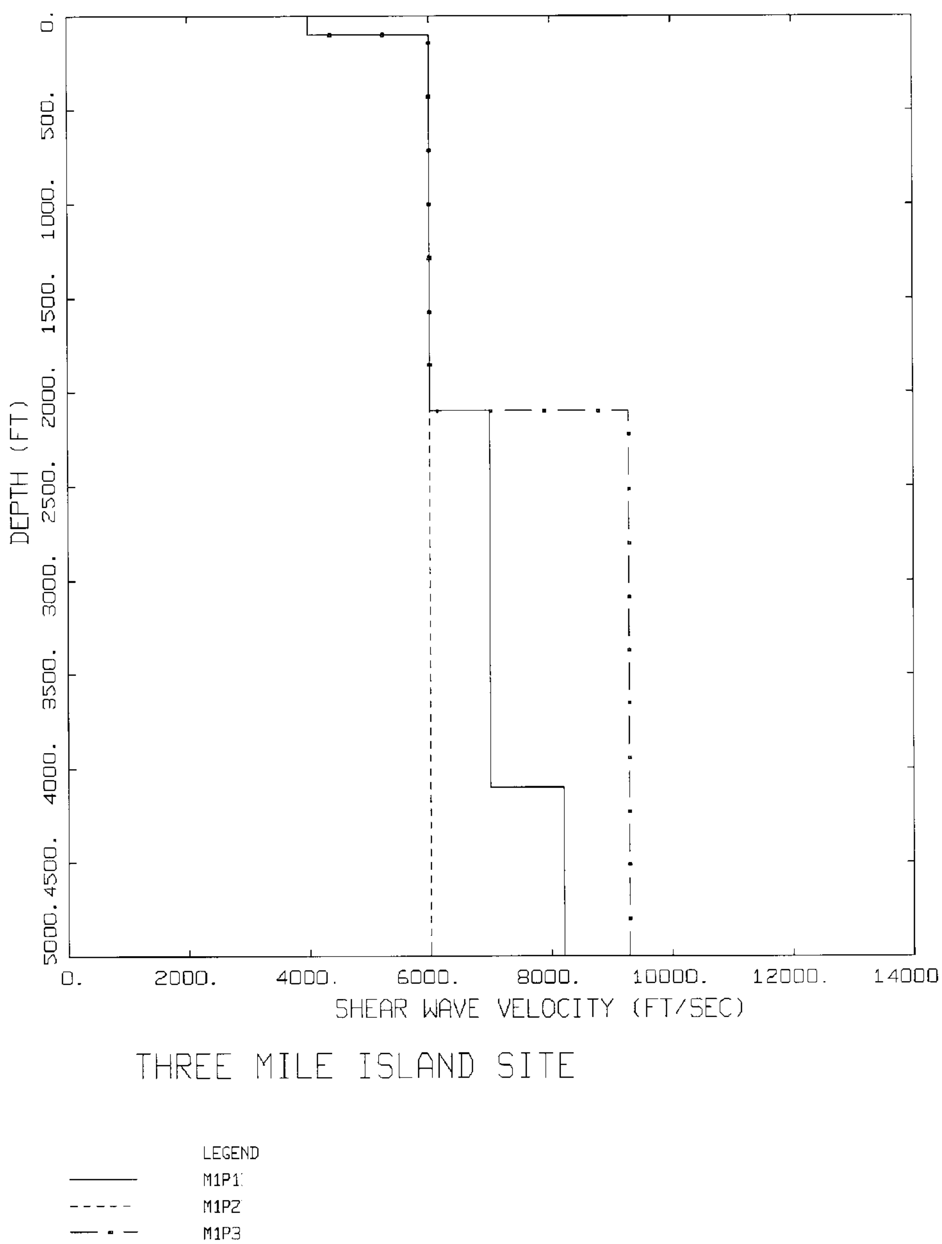

Figure B-14
Shear-Wave Velocity Profiles for the Three Mile Island Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate lower and higher velocities at depth. All three profiles are estimates below a depth of about 50 ft (15m).

B.14.2.2 Modulus Reduction and Hysteretic Damping Curves

Peninsular Range rock curves (Silva et al., 1997) are used over the top 100 ft (31m) in all three profiles (Figure B-14).

B.14.2.3 Regional Crustal Damping (kappa)

For a shear-wave velocity of 4,000 ft/sec over the top 100 ft, Equation B-1 gives a kappa value of about 0.02 sec, which was adopted as the base-case value (Table B-14). To accommodate higher and lower kappa values, 0.04 sec and 0.01 sec were considered as well. The higher kappa value, 0.04 sec considers the site as having typical western North America soft rock conditions (Anderson and Hough 1984; Silva et al., 1997) while the low kappa (0.01 sec) reflects an assumption of firm to hard rock conditions (Silva and Darragh, 1995).

B.14.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-14.

Table B-14
Three Mile Island Weights

Properties*	**Category Weights**
M1P1	0.5
M1P1.KH	0.2
M1P1.KL	0.3
M1P1	0.6
M1P2	0.2
M1P3	0.2
	Combined Weights
M1P1	0.3
M1P1.KH	0.2
M1P1.KL	0.3
M1P2	0.1
M1P3	0.1

*M1P1; base case profile, kappa = 0.02 sec

M1P1.KH; base case profile, kappa = 0.04 sec

M1P1.KL; base case profile, kappa = 0.01 sec

M1P2; low deep velocity profile, kappa = 0.02 sec

M1P3; high deep velocity profile, kappa = 0.02 sec

B.15 VOGTLE SITE

The Vogtle Nuclear Plant is located on the southwest side of the Savannah River in Burke County, Georgia across the river from Barnwell County, South Carolina. The site is in the Tifton Upland of the Atlantic Coastal Plain physiographic province.

The near surface deposits are Quaternary alluvial deposits from the Savannah River and its tributaries. The Blue Bluff Member of the middle Eocene Lisbon formation forms the foundation for critical plant structures. This moderately hard calcareous siltstone or marl is underlain by quartz sand. The total thickness of the Blue Bluff member at the plant site is about 70 ft (21m). The quartz sand is about 100 ft (30m) thick and overlies an approximately 50 ft (15m) thick unit composed of interbedded clay, silty sand, and lignitic beds representing the Huber and Ellenton Formations of Paleocene age (EPRI, 1989).

The pre-Tertiary units include approximately 600 ft (180m) of Cretaceous sediments including the Tuscaloosa Formation that consists of fluvial and estuarine deposits of sand and minor gravel intercalated with silt and clay. The contact between the Cretaceous and the basement complex is at a depth exceeding 1,000 ft (305m) below the surface. The basement complex includes sediments of the buried Triassic Dunbarton Basin that mainly consist of breccias in a matrix of claystone and siltstone, alternating layers of sandstone, mudstone, siltstone, and claystone. The Precambrian crystalline basement rocks exposed northwest of the site include gneiss, granite, phyllite and greenstone (EPRI, 1989).

B.15.1 Soil Profile Information

The containment buildings are founded on compacted select sand backfill 61 ft (18.6m) below plant grade with the base of the reactor cavity mat at a depth of 85 ft (26m). The fill was placed in an excavation 90 ft (27m) below finished grade resulting in about 28.5 ft (8.7m) of fill below the containment building foundation. The excavation was made because the original soil consisted of very loose to dense sands that were potentially liquefiable and due to the presence of the thin shelly Utley limestone. A very hard calcareous clay marl (Blue Bluff Member of the Lisbon Formation) about 70 ft (21m) thick underlies the fill. A thick (750 ft (229m)) dense, coarse sand with minor interbedded silty clay and clayey silts underlies the marl. Basement rock is estimated to be at a depth of about 1,500 ft (457m) (rock hazard defined as basement material with a Vs of 2.83 km/sec).

Compressional- and shear-wave velocity measurements were made from cross-hole tests to a depth of 290 ft (88m). At this depth the shear-wave velocity of the clay marl was 1,700 ft/sec (520 m/sec). The shear-wave velocity for the compacted fill is calculated from shear moduli and density to be about 767 ft/sec (234 m/sec) in the top 10 ft (3m) and 1258 ft/sec (384 m/sec) below 10 ft (3m).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.15.2 Description of Base Case Profiles

B.15.2.1 Shear Wave Velocity Profiles

The base-case shear-wave velocity profile reflecting reactor embedment conditions consists of about 30 ft (9m) of compacted fill overlying about 210 ft (64m) of measured (crosshole) velocities. Below that depth (about 240 ft (73m)), the nearby Savannah River Site profile (SRS, 1996) was adopted to a depth of 1,000 ft (305m). At this depth the Triassic Dumbarton basin was assumed to overly Precambrian basement, taken at a depth of about 1,500 ft (457m). Depth to the Dumbarton Basin sedimentary material was randomized ± 400 ft (122m).

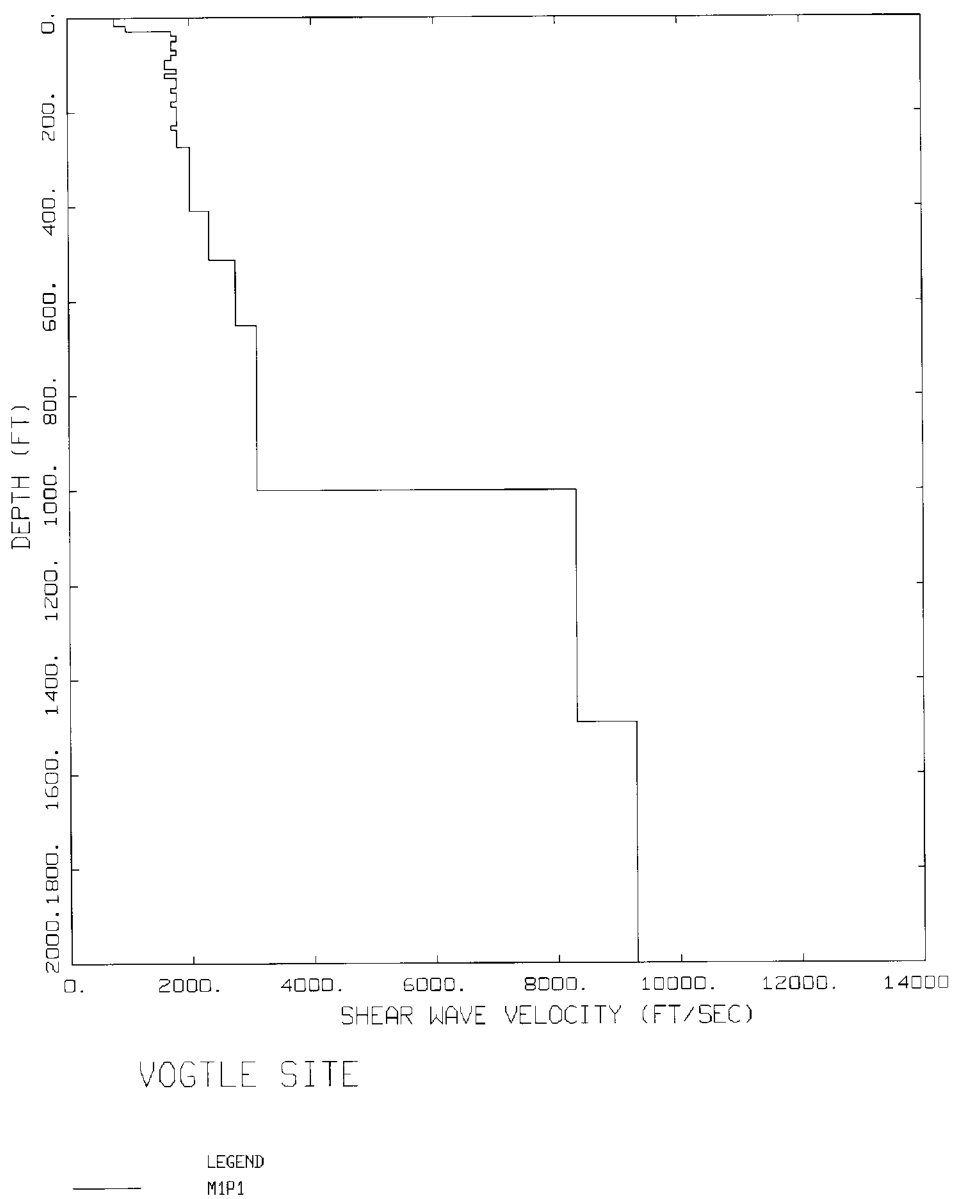

Figure B-15
Shear-Wave Velocity Profiles for the Vogtle Site

B.15.2.2 Modulus Reduction and Hysteretic Damping Curves

For the fill material (top 30 ft (9m)), the EPRI (1993) curves were used throughout. To accommodate epistemic variability in appropriate suites of curves for the profile below the fill, three sets of G/G_{max} and hysteretic damping curves were used: EPRI (1993), Peninsular Range (Silva et al., 1997; 1998b), and Savannah River (SRS, 1996). For the base-case profile (M1P1) the Savannah River curves were used over the top 500 ft (152m), beginning with the Savannah River shallow clay curves (SRS, 1996), taken to occur below the fill at the Vogtle site. Below a depth of 500 ft, the soil is assumed to behave in a linear manner (Silva et al., 1997; 1998b). Profile M2P1 considers the same shear-wave velocities with the EPRI (1993) curves replacing Savannah River curves below the fill and to the depth where the Savannah River profile was added below the Vogtle measured velocities (about 240 ft (73m) in Figure B.15).

Profile M3P1 replaces the EPRI (1993) curves below the fill material with Peninsular Range curves while profile M4P1 has Savannah River curves below the fill and EPRI (1993) below the portion of the Vogtle profile with measured shear-wave velocities. For the Savannah River curves, the Savannah River shallow clay was used for the Vogtle marls. The entire suite of dynamic material model combinations is listed in Table B-15.

B.15.2.3 Regional Crustal Damping (kappa)

Based on a measured kappa value at the nearby Savannah River Site of 0.02 sec (Fletcher, 1995), the base-case total site kappa value was assumed to be 0.02 sec. A 50% increase to 0.03 sec was taken as the high value (M1P1.KH). For a low kappa (M1P1.KL), the base-case profile (M1P1), with a low-strain damping contributing a kappa value of 0.0064 sec, was added to the hard rock kappa of 0.006 sec and rounded up to total 0.013 sec, to accommodate 426 ft (130m) of Triassic Basin sedimentary rock overlying hard rock conditions.

B.15.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-15.

Table B-15
Vogtle Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.2
M1P1.KL	0.2
M1	0.7
M2, M3, M4, M5	0.3
	Combined Weights
M1P1	0.420
M1P1.KH	0.140
M1P1.KL	0.140
M2P1	0.075
M3P1	0.075
M4P1	0.075
M5P1	0.075

*M1P1; base case profile, kappa = 0.02 sec

M1P1.KH; base case profile, kappa = 0.03 sec

M1P1.KL; base case profile, kappa = 0.013 sec

M2P1; base case profile, kappa = 0.02 sec

M3P1; base case profile, kappa = 0.02 sec

M4P1; base case profile, kappa = 0.02 sec

M5P1; base case profile, kappa = 0.02 sec

Table B-16
Vogtle G/G_{max} and Hysteretic Damping Curves

Profiles					
Depth	M1P1	M2P1	M3P1	M4P1	M5P1
0 ft – 30 ft (9m)	EPRI	EPRI	EPRI	EPRI	EPRI
30 ft – 240 ft (9m – 73m)	SR	EPRI	PR	SR	SR
240 ft – 500 FT (73m – 152m)	SR	SR	SR	EPRI	PR

where

– EPRI represents EPRI (1993) curves

– SR represents Savannah River curves (SRS, 1996)

– PR represents Peninsular Range curves (Silva et al., 1997, 1998b)

B.16 WATERFORD SITE

The Waterford Steam Electric Station is located in southern Louisiana within the Mississippi River deltaic plain physiographic province. Since early Jurassic time, nearly continuous marine deposition has resulted in strata in excess of 40,000 ft (12,200m) beneath the site.

The upper 500 ft (152m) of the site has been characterized by drilling as flat lying sediments. The top 53 ft (16m) consists of recent clays and silty clay with silt and sand lenses. The Pleistocene Prairie Formation consists of interbedded sands and clays with varying amounts of silt and extends to a depth of about 1,100 ft (335m). The Pliocene – Pleistocene deposits consist of the Citronelle Formation of interbedded sands and clays that extend to about 1,900 ft (580m). Beneath these strata are about 3,000 ft (915m) of Pliocene clays with relatively thin sand layers. Between 7,500 and 10,500 ft (2,285 to 3,200m) is a sequence of shale alternating with thin sandstone layers. This unit overlies a continuous sequence of shale ranging in age from middle to upper Jurassic. The lower Jurassic Louann salt beds are the deepest sediments known to occur above crystalline bedrock. Precambrian crystalline basement rock was estimated to be at a depth greater than 40,000 ft (12,200m) (EPRI, 1989).

B.16.1 Soil Profile Information

The reactor building is founded upon Pleistocene stiff, tan, gray, and fissured clay at a depth of about 60 ft (18m) below natural grade. The thickness of this stratum is approximately 37 ft (11m) (30 ft (9m) below reactor foundation). This layer is underlain by a 15 ft (4.6m) thick soil of very dense silty sand and then by over 100 ft (30m) of clay layers with various stiffnesses. Hard rock hazard is defined as basement material with a Vs of 2.83 km/sec.

Geophysical measurements, including seismic uphole and cross-hole, were performed. In the deepest boring, shear-wave velocities around 1,075 ft/sec (330 m/sec) are measured at depths of about 220 ft (67m) in the Pleistocene soils. These measurements were projected into the lower

Pleistocene at depth of 400 ft (122m) with a shear-wave velocity of 1,625 ft/sec (495 m/sec) in EPRI (1989).

Site-specific laboratory dynamic material testing for modulus reduction and hysteretic damping strain dependencies reflecting recent procedures were not available for this site.

B.16.2 Description of Base Case Profiles

B.16.2.1 Shear Wave Velocity Profiles

The base-case shear-wave velocity profile (M1P1) is shown in Figure B-16 and is based on measured shear-wave velocities to a depth of about 250 ft (76m). The profile reflects embedment depths at the surface (top 60 ft (18m) removed from the site profile). Below that depth the measured velocity was extended 200 ft (61m) and the uplands profile (see Grand Gulf, Section B.5) developed for the upper Mississippi embayment was added to a depth of 2,500 ft (762m). This depth was considered sufficient to capture soil amplification to the lowest frequency of interest (0.5 Hz).

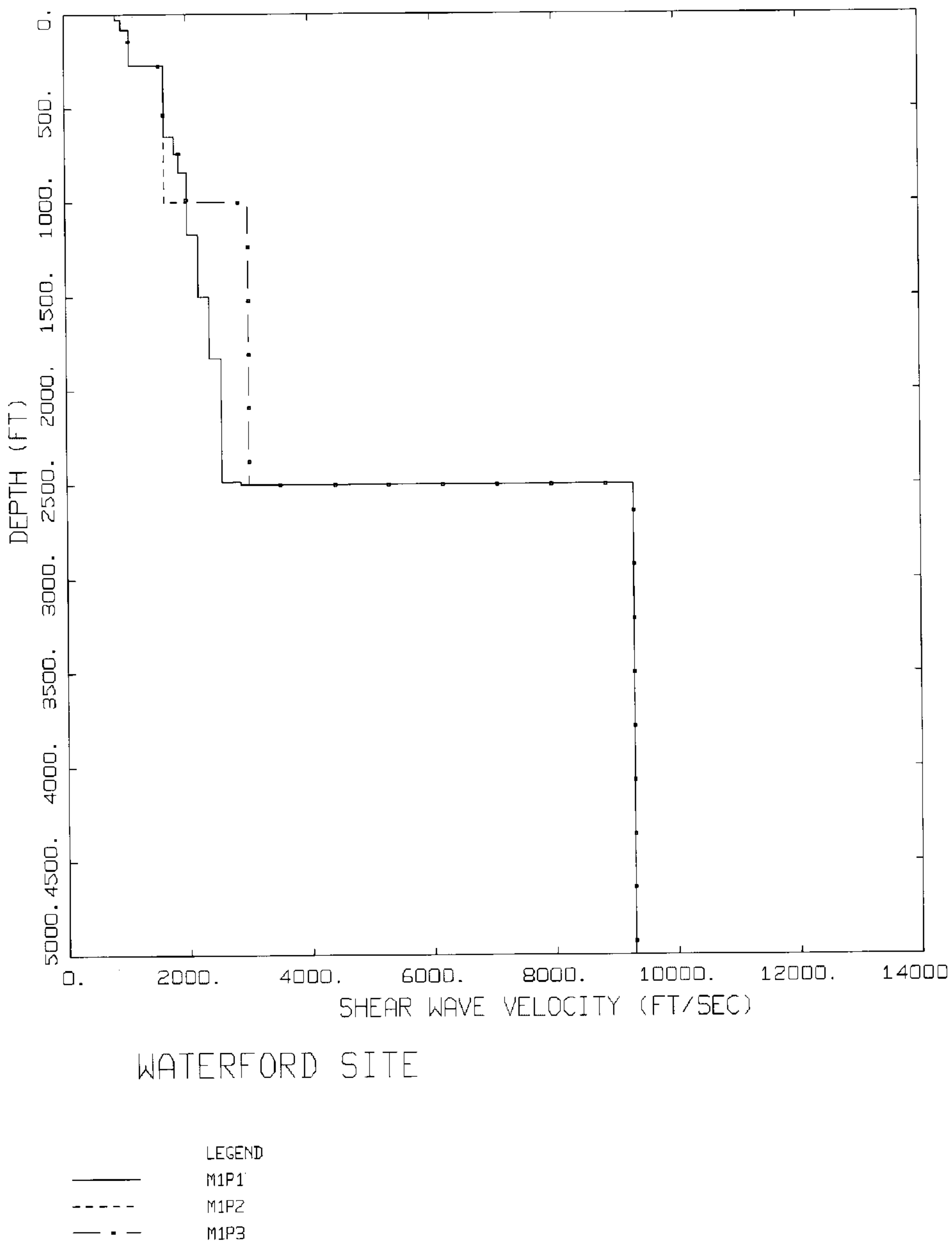

Figure B-16
Shear-Wave Velocity Profiles for the Waterford Site

Profile M1P1 is considered the base case profile with M1P2 and M1P3 to accommodate lower and higher velocities at depth. All three profiles are estimates below a depth of about 250 ft (76m).

Alternative profiles were considered as well with profile M1P2 extending the deepest measured shear-wave velocity (1,625 ft/sec (495 m/sec)) to a depth of 1,000 ft (305m) where it merges with the base-case profile (Mississippi embayment uplands generic profile). To consider higher at-depth velocities, profile M1P3 (Figure B-16) has an increase in velocity to 3,000 ft/sec (914 m/sec) at a depth of 1,000 ft (305m). All three profiles have depth to hard rock randomized $\pm$ 1,000 ft (305m).

B.16.2.2 Modulus Reduction and Hysteretic Damping Curves

To provide alternative base-case dynamic material properties, both the EPRI (1993) (M1P1) and Peninsular Range (Silva et al., 1997, 1998b) (M2P1) curves were considered. EPRI (1989) shows G/G_{max} and hysteretic damping curves based on laboratory dynamic material testing. Although these curves do not reflect more recent procedures and are not considered reliable for direct use, they do suggest the EPRI (1993) curves may be more appropriate for these materials and the amplification weights (Table B-17) have been selected to reflect these considerations.

B.16.2.3 Regional Crustal Damping (kappa)

As with the other Mississippi embayment sites (Grand Gulf, Section B.5 and River Bend, Section B.10) the total site kappa value for the base-case was 0.046 sec. High and low kappa values based on a $\pm$ 50% variation result in 0.069 sec (M1P1.KH in Table B-17) and 0.031 sec (M1P1.KL in Table B-17) respectively.

B.16.2.4 Profile Weights

The profile weights for the amplification factors are listed below in Table B-17.

Table B-17
Waterford Weights

Properties*	Category Weights
M1P1	0.6
M1P1.KH	0.2
M1P1.KL	0.2
M1P1	0.8
M1P2	0.1
M1P3	0.1
M1	0.6
M2	0.4
	Combined Weights
M1P1	0.288
M1P1.KH	0.096
M1P1.KL	0.096
M1P2	0.060
M1P3	0.060
M2P1	0.400

*M1P1; base case profile, EPRI Curves, kappa = 0.046 sec

M1P1.KH; base case profile, EPRI Curves, kappa = 0.069 sec

M1P1.KL; base case profile, EPRI Curves, kappa = 0.031 sec

M1P2; low gradient profile, EPRI Curves, kappa = 0.046 sec

M1P3; high gradient profile, EPRI Curves, kappa = 0.046 sec

M2P1; base case profile, Peninsular Range Curves, kappa = 0.046 sec

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