J. A. "Buzz" Miller
Senior Vice President
Nuclear Development

Southern Nuclear
Operating Company, Inc.
40 Inverness Center Parkway
Post Office Box 1295
Birmingham, Alabama 35201

Tel 205.992.5754
Fax 205.992.6165

SOUTHERN COMPANY
Energy to Serve Your World℠

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Southern Nuclear Operating Company
Vogtle Early Site Permit Application
Supplement To Provide Additional Seismic Analysis

Ladies and Gentlemen:

By letter AR-08-0483, dated March 28, 2008, Southern Nuclear Operating Company (SNC) submitted Revision 4 of the Vogtle Early Site Permit (ESP) Application to the U.S. Nuclear Regulatory Commission (NRC). Based on subsequent discussions with the NRC regarding geotechnical information presented in Section 2.5, *Geology, Seismology, and Geotechnical Engineering*, of the ESP application's Site Safety Analysis Report (SSAR), SNC is providing supplemental information for two subsections in SSAR Section 2.5. The supplemental information provides additional detail and clarification concerning the supporting analysis. The two supplemented SSAR subsections (2.5.2.9.3, *Updated Site Response Analyses*, and 2.5.4.7, *Response of Soil and Rock to Dynamic Loading*) are provided in the enclosure to this letter. The revision numbers for the SSAR pages containing the supplemented application subsections have been changed to "4-S1" to distinguish them from their existing revision numbers.

This ESP application supplement does not contain restricted data or other defense information that requires separation from the unclassified information in accordance with 10 CFR 50.33(j) pursuant to 10 CFR 52.17(a)(1).

The SNC licensing contact for the supplemental information is J. T. Davis at (205) 992-7692.

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NRO

Mr. J. A. (Buzz) Miller states he is a Senior Vice President of Southern Nuclear Operating Company, is authorized to execute this oath on behalf of Southern Nuclear Operating Company and to the best of his knowledge and belief, the facts set forth in this letter are true.

Respectfully submitted,

SOUTHERN NUCLEAR OPERATING COMPANY

Joseph A. (Buzz) Miller

Sworn to and subscribed before me this 28 *day of* April*, 2008*

Notary Public: Gloria H. Brie

My commission expires: 05/06/09

JAM/BJS/dmw

Enclosure: Vogtle Early Site Permit Application Supplement 4-S1

<u>Southern Nuclear Operating Company</u>
Mr. J. B. Beasley, Jr., President and CEO (w/o enclosure)
Mr. J. T. Gasser, Executive Vice President, Nuclear Operations (w/o enclosure)
Mr. T. E. Tynan, Vice President - Vogtle (w/o enclosure)
Mr. D. M. Lloyd, Vogtle Deployment Director (w/o enclosure)
Mr. C. R. Pierce, Vogtle Development Licensing Manager (w/o enclosure)
Mr. D. P. Moore, Engineering Programs Consulting Engineer
Document Services RTYPE: AR01
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<u>Nuclear Regulatory Commission</u>
Mr. R. W. Borchardt, Director of Office of Nuclear Regulation (w/o enclosure)
Mr. V. M. McCree, Acting Regional Administrator
Mr. D. B. Matthews, Director of New Reactors (w/o enclosure)
Ms. S. M. Coffin, AP1000 Manager of New Reactors (w/o enclosure)
Mr. C. J. Araguas, Project Manager of New Reactors
Mr. M. D. Notich, Environmental Project Manager
Mr. W. F. Burton, Chief – Environmental Technical Support (w/o enclosure)
Mr. G. J. McCoy, Senior Resident Inspector of VEGP

<u>Georgia Power Company</u>
Mr. O. C. Harper, Vice President, Resource Planning and Nuclear Development (w/o enclosure)

<u>Oglethorpe Power Corporation</u>
Mr. M. W. Price, Chief Operating Officer (w/o enclosure)

<u>Municipal Electric Authority of Georgia</u>
Mr. C. B. Manning, Senior Vice President and Chief Operating Officer (w/o enclosure)

<u>Dalton Utilities</u>
Mr. D. Cope, President and Chief Executive Officer (w/o enclosure)

<u>Burke County Chief Executive</u>
Mr. James M. Dixon, Chairman, Burke County Board of Commissioners

Southern Nuclear Operating Company

AR-08-0676

Enclosure

Vogtle Early Site Permit Application

Supplement 4-S1*

NOTE: This enclosure contains Supplement 4-S1 pages for the following ESPA SSAR sections:

- Section 2.5.2.9.3 Updated Site Response Analyses (5 pages)
- Section 2.5.4.7 Response of Soil and Rock to Dynamic Loading (17 pages)

* ESP Application Part 2 Title Sheet, Table of Contents and List of Figures that have been affected by this application supplement are not included. These application documents will be revised in the next application revision, as applicable.

2.5.2.9.3 Updated Site Response Analyses

As discussed in Section 2.5.4.7, additional geotechnical data was collected as part of COL investigation and the ESP data was supplemented with the new data. The completed data set is referred to as "COL" data. Section 2.5.4.7.1 presents a discussion of the shear wave velocity profile of the ESP and COL data sets and Section 2.5.4.7.2 presents a discussion of the strain-dependent soil properties of the two data sets. Site specific (COL) strain-dependant soil properties are presented in Figures 2.5.4-9a and 2.5.4-11a and in Table 2.5.4-12a. Section 2.5.4.7.5 presents the comparison of the data in terms of the shear wave velocity profile (Figure 2.5.4-7a) and strain-dependent soil properties (Figures 2.5.4-19a through 2.5.4-20c). Soil amplification analysis and development of FIRS described in Section 2.5.2.5.1 are based on the ESP data. In this section, the effect of COL data on the soil amplification at the depth of 40 ft (FIRS) is presented. The FIRS at 40 ft depth has been used as input for the site specific evaluation of the AP1000 design.

As described in Section 2.5.2.5.1, development of soil amplification using ESP data is based on 60 randomized velocity profiles and associated EPRI and SRS strain-dependent soil properties incorporating 30 time histories for the HF and LF motions at each MAFE of 10^{-4}, 10^{-5}, and 10^{-6} levels. However for the purpose of sensitivity analysis and evaluation of the effects of COL data, a limited number of soil column analysis have been performed as described below.

Using the best estimate COL velocity profile (Figure 2.5.4-7a), the upper bound and lower bound profiles were developed using a variation of the data set. The three COL velocity profiles and the associated COL strain-dependent soil properties were analyzed using three HF and three LF time histories corresponding to MAFE of 10^{-4}. All analyses were performed twice to consider the low and high PI strain-dependent soil properties for BBM (Figures 2.5.4-19b and 2.5.4-20b). Several iterations were performed in each run to converge on the soil properties. The strain-compatible velocity profiles and damping profiles obtained from the analysis are shown in Figure 2.5.2-65a and 2.5.2-65b labeled as COL. From each run, the response motion in terms of 5% acceleration response spectrum at the depth of 40 ft as SHAKE "outcrop" motion was computed. The results of HF input motion were averaged over the three time histories and over the three soil profiles as well as the low and high PI cases of the BBM. The same averaging method was used for the LF input motions. The averaged results were divided by the corresponding HF and LF input response spectrum at MAFE of 10^{-4} to compute the spectral amplification at the 40 ft horizon. The resultant two amplifications curves were enveloped. The enveloped amplification values are shown in Figure 2.5.2-65c labeled as COL.

To provide a consistent soil amplification for comparison with the results using COL data, from the ESP set of runs described in Section 2.5.2.5.1, the strain-compatible velocity and damping profiles were used to obtain the median and upper and lower bound profiles (using one standard deviation as the variation). The velocity and damping profiles are compared with the corresponding profiles from the analysis of the COL data in Figures 2.5.2-65a and 2.5.2-65b, respectively. Except for the damping profile at shallow depths, the two sets of data are consistent. The three profiles selected from the analysis of the ESP data were subsequently analyzed using the same three HF and LF time histories used in the analysis of the COL data. Since the soil properties are already compatible with shear strains, no further iteration on soil properties was performed. The results in terms of acceleration response spectrum at 5% damping at 40 ft depth as outcrop motion were obtained. The spectra for each HF and LF motions were averaged over the three time histories and over the three profiles. The averaged responses were divided by the respective HF and LF response spectra at MAFE of 10^{-4} and the resultant amplifications were enveloped. Figure 2.5.2-65c shows the amplification labeled as ESP.

To confirm the adequacy of the limited number of profiles and time histories for the purpose of this evaluation, the amplification corresponding to the analyses of the fully randomized ESP soil profiles (Section 2.5.2.) is also shown in Figure 2.5.2.65c labeled as "ESP-all". The comparison of the two sets of results based on ESP data shows the selection of limited number of profiles and time histories are adequate for the purpose of the evaluation of the impact of the COL data. Furthermore, the comparison of the amplification between ESP and COL data is considered small and is expected to be reduced to be negligible if the fully randomized soil profiles were used in the COL set of analyses.

In addition, an assessment of the small differences in the amplification of the FIRS motion on the structural response of the AP1000 has been made. The AP1000 has significant margin when compared to Vogtle site specific seismic floor response spectra associated with the ESP and sensitivity soil profiles. Based on this evaluation, it has been determined that the AP1000 certified Design remains acceptable for the Vogtle site.

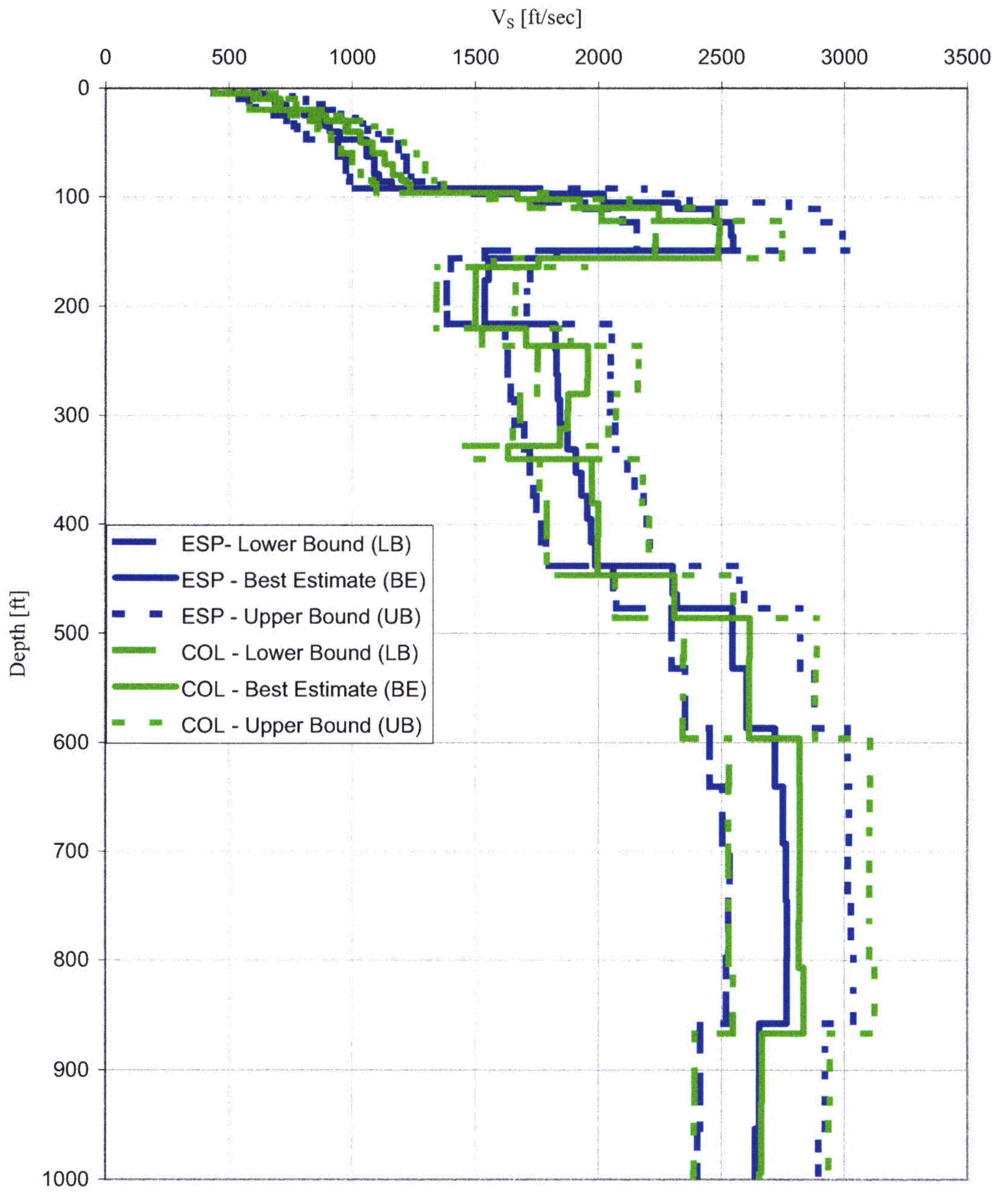

Figure 2.5.2-65a Vogtle Strain Compatible Profiles (S-Wave Velocity)

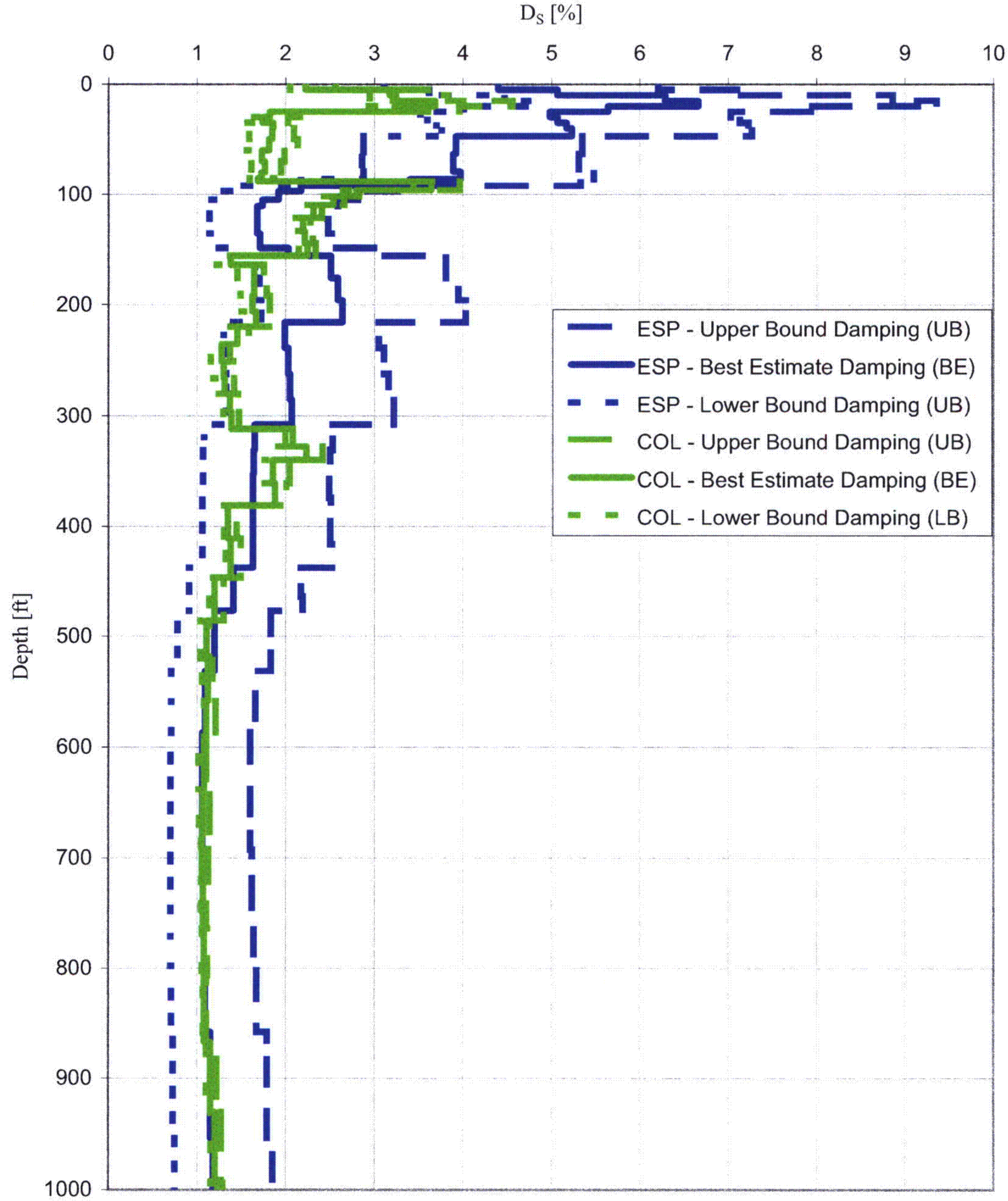

Figure 2.5.2-65b Vogtle Strain Compatible Profiles (S-Wave Damping)

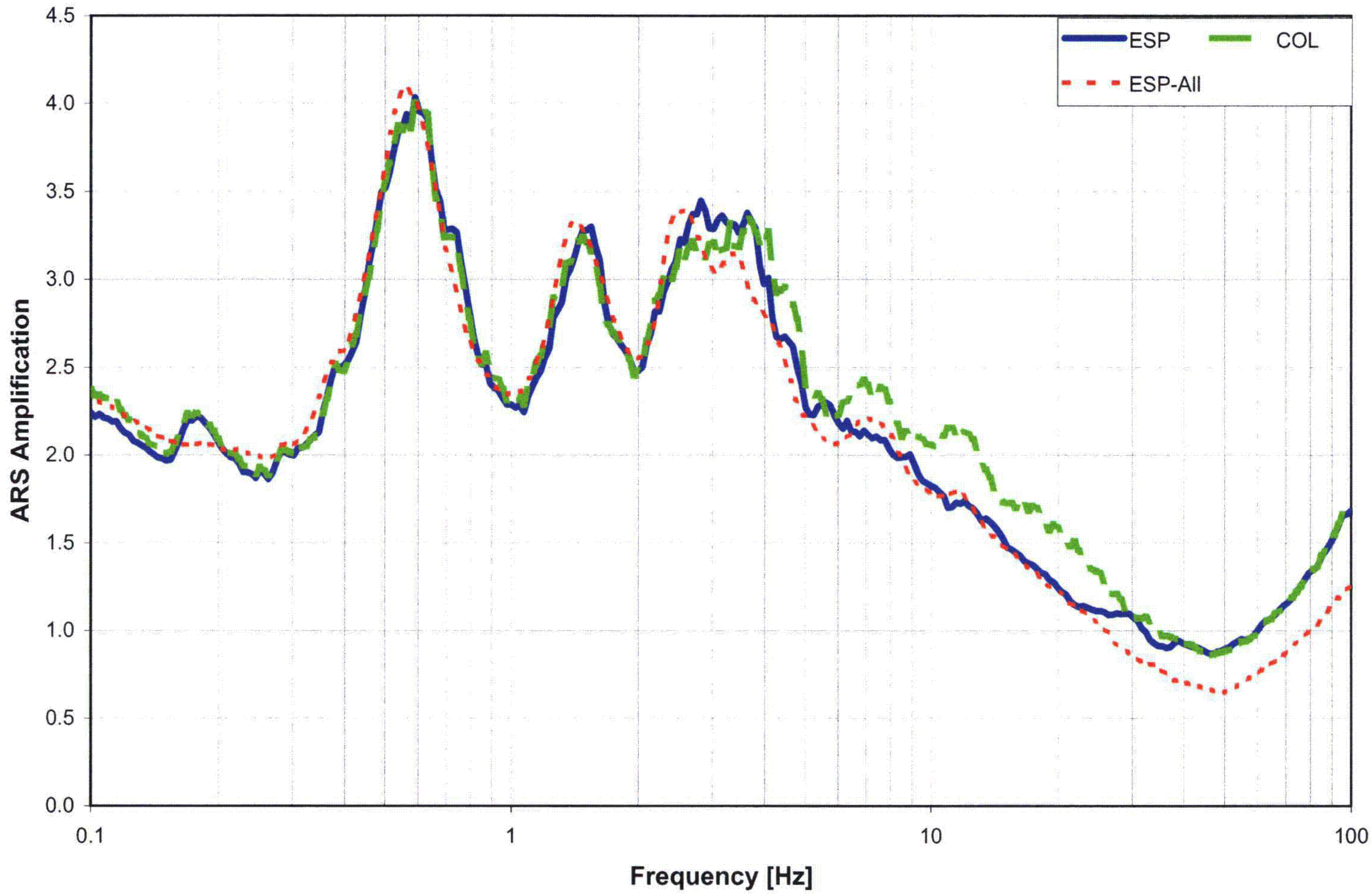

Figure 2.5.2-65c Envelope at 40 ft depth (FIRS horizon)

2.5.4.7 Response of Soil and Rock to Dynamic Loading

All new safety-related structures will be founded on the planned structural backfill, which will completely replace the existing Upper Sand Stratum soils. The seismic acceleration at the sound bedrock level will be amplified or attenuated up through the soil and rock column. To estimate this amplification or attenuation, the following data are required.

- Shear wave velocity profile of the soils and rock
- Variation with strain of the shear modulus and damping values of the soils
- Site-specific seismic acceleration-time history

In addition, an appropriate computer program is required to perform the analysis.

2.5.4.7.1 Shear Wave Velocity Profile

2.5.4.7.1.1 Soil Shear Wave Velocity Profile

Various measurements have been made at the VEGP ESP site to obtain estimates of the shear wave velocity in the soil. Measurements were also made at the site during the COL investigation to confirm ESP estimates of shear wave velocity in the soil.

All safety-related structures will be founded on the structural backfill that will be placed on top of the Blue Bluff Marl after complete removal of the Upper Sand Stratum. Shear wave velocity was not determined for the compacted backfill during the ESP subsurface investigation. Data for existing Units 1 and 2 is used **(Bechtel 1984)**, and the backfill shear wave velocity values are summarized in Table 2.5.4-10.

During the COL investigation, shear wave velocity data for the compacted backfill was measured directly in the field during the Phase I test pad program. These data, with laboratory test data, were used to evaluate the shear wave velocity of the backfill. A summary of the Phase I test pad program, including a discussion of material properties, is included in Section 2.5.4.5.3. The results of the test pad program are presented in Appendix 2.5D. RCTS and other data from the COL investigation were also used to evaluate the shear wave velocity of the backfill. The RCTS data are presented in Attachment G of Appendix 2.5C. Results of the COL investigation and Phase I test pad were used to develop the shear wave velocity profile of the backfill based on COL data. This profile is presented in Table 2.5.4-10a and is in good agreement with the ESP backfill profile. Both of these profiles are included in the respective soil columns in Figure 2.5.4-7a.

Figure 2.5.4-6 shows the shear wave velocity values measured in the subsurface soil and rock strata for the ESP subsurface exploration program using suspension P-S velocity and CPT down-hole seismic testing. Figure 2.5.4-6a shows the shear wave velocity values measured in the Upper Sand Stratum using CPT down-hole seismic testing from COL data. The shear wave velocity profile shown in Figure 2.5.4-7 is the profile interpreted from the results of the ESP data shown in Figure 2.5.4-6 for strata below the Upper Sand Stratum, plus the shear wave velocity values for the backfill shown on Table 2.5.4-10. The shear wave velocity values corresponding to the profile shown on Figure 2.5.4-7 for the different soil strata encountered by the borings are provided in Table 2.5.4-11.

The shear wave velocity profile developed from the ESP investigation and shown in Figure 2.5.4-7 is used in the seismic amplification/attenuation analysis. The soil profile used consists of: Compacted backfill from 0 to 86 ft, Blue Bluff Marl from 86 to 149 ft, Lower Sand Stratum from 149 to 1,049 ft, Dunbarton Triassic Basin and Paleozoic Crystalline Rock below 1,049 ft.

During the COL investigation, shear wave velocity values were measured in the Blue Bluff Marl and the upper portions of the Lower Sand Stratum as previously described in Section 2.5.4.4.3. These data included measurements in 6 boreholes, extending to a maximum depth of 420 feet below ground surface. Shear wave velocity values were measured in the Still Branch, Congaree, and Snapp Formations of the Lower Sand Stratum. These COL data (6 profiles) were combined with two ESP profiles (located in the powerblock area of Units 3 and 4) and averaged. The average shear wave profile for the this COL data set is shown on Figure 2.5.4-7a. This profile also reflects the average stratigraphy within the powerblock excavation footprints based on data from the COL borings. The shear wave velocity profile includes the shear wave velocity profile of the backfill that was developed during the Phase I test pad program. The profile below the COL data (below the upper portion of the Snapp formation) incorporates the shear wave velocity data from the ESP profile. The COL profile consists of: compacted backfill from 0 to 88 ft, Blue Bluff Marl from 88 to 156 ft, Lower Sand Stratum from 156 to 1,058 ft, and Dunbarton Triassic Basin and Paleozoic Crystalliine Rock below 1,058 ft. The ESP profile, shown on Figure 2.5.4-7, is also illustrated on Figure 2.5.4-7a for comparison purposes. Figure 2.5.4-7a illustrates the relationship, including the similarity, between the two data sets. In general, within specific geologic formations, the two profiles demonstrate consistent shear wave velocity characteristics. The profile of the combined data set (COL) in the middle and upper portions of the Blue Bluff Marl is in good agreement with the ESP profile. At the lower portions of the Blue Bluff and in the Lower Sand Stratum, the COL profile exhibits slightly lower shear wave values than the ESP profile.

2.5.4.7.1.2 Rock Shear Wave Velocity Profile

As discussed in Section 2.5.4.2.2, the VEGP ESP site sits on over 1,000 feet of Coastal Plain sediments underlain by Triassic Basin sedimentary rock, which in turn is underlain by Paleozoic crystalline rock (see Figure 2.5.1-40). For the purpose of subsequent site response analysis, for which input rock time histories must be inserted at a depth where the material shear-wave velocity is approximately 9,200 ft/s, it is necessary to know the shear-wave velocity profile and materials properties for the site down to the depth at which this velocity is encountered. Because the site overlies both Triassic Basin and Paleozoic crystalline rocks, it is necessary to consider effect of shear-wave velocities and material properties of both rock types and their geometries.

As indicated in Figure 2.5.4-6, the shear-wave velocities measured at the top of the Triassic Basin, even through the weathered portion, do not reach the velocity of 9,200 ft/s. Inspection of available deep borehole shear-wave velocity at SRS **(SRS 2005)** along with the B-1003 data [Figure 2.5.4-8], however, suggests the following character of rock shear-wave in the Triassic Basin:

- A weathered zone of ~200 feet thickness occurs at the top of the Triassic Basin, characterized by a steep shear-wave velocity gradient, where the shear-wave velocity rapidly increases with depth to a point where a relatively high shear-wave velocity, but less than 9,200 ft/s is reached;
- Below the weathered zone the shear-wave velocity increases with a gentler gradient within the unweathered rock;
- Considering the SRS data as a guide for shear-wave velocity within deep portions of the Triassic Basin, there are a range of gentle gradients and a range of shear-wave velocities for the top of the unweathered Triassic Basin that could be considered as a continuation of the site-specific profile presented by B-1003.

Figure 2.5.1-41 indicates that the non-capable Pen Branch fault separates the Triassic Basin from the Paleozoic crystalline rocks. The structural geometry of these rock units and the fault, relative to the locations of boreholes B-1002 and B-1003 (approximate locations of the proposed nuclear units) and considering the velocity profiles shown in Figure 2.5.4-8, a shear-wave velocity profile through the Triassic Basin would not likely reach 9,200 ft/s before encountering the Paleozoic crystalline rock. Several observations and studies at SRS [e.g., (**Geovision 1999**, **Lee et al 1997**, **Domaracki 1994**)] indicate that the shear-wave velocity of the Paleozoic crystalline rock is at least 9,200 ft/s.

Therefore, to represent the variability of the depth at which the Paleozoic crystalline rock is encountered, with a shear-wave velocity of at least 9,200 ft/s, and the uncertainty of the shear-wave velocity gradient and velocity at the top of the unweathered Triassic Basin, six rock shear-wave velocity profiles were considered to comprise the base case used in the seismic amplification/attenuation analysis. Figure 2.5.4-7 shows a plot of these six rock shear-wave velocity profiles and Table 2.5.4-11, Part B presents their tabulation.

Figures 2.5.1-40 and Figure 2.5.4-8 suggest additional geometries for the shear-wave velocity profiles of the Triassic Basin and the Paleozoic crystalline rock that could impact site response. As interpreted in Figure 2.5.1-41, further to the northwest of the footprint of the project site the coastal Plain sediments would be underlain immediately by the Paleozoic crystalline rock. Conversely, further to the southeast of the footprint of the project, the Paleozoic crystalline rock is at such a depth that the shear-wave velocity gradient in the Triassic Basin would result in 9,200 ft/s being reached in the shear-wave velocity profile while still within the Triassic Basin. Close inspection of the DRB-9 shear-wave velocity profile in Figure 2.5.4-8 suggests a low-velocity zone at the bottom of the Triassic Basin at the encountering of the Pen Branch fault. Sensitivity analyses were performed that indicated that alternate shear-wave velocity models suggested by these observations result in insignificant variations in the site response, relative to the six profiles that were explicitly considered, as discussed above.

2.5.4.7.2 Variation of Shear Modulus and Damping with Shear Strain

2.5.4.7.2.1 Shear Modulus

2.5.4.7.2.1.1 ESP Analysis

The variation of soil shear modulus values of sands, gravels, and clays with shear strain is well-documented by researchers such as Seed and Idriss (1970); Seed et al. (1984); and Sun et al. (1988). This research, along with additional work, has been summarized by EPRI **(EPRI TR-102293 1993)**.

Shear modulus is derived from the respective unit weight and shear wave velocity of the soil strata with the following equation:

$$G_{max} = \rho \cdot (V_s)^2 = \gamma \cdot (V_s)^2/g \qquad \text{Equation (20-27) on page 758 of Bowles (1982)}$$

Shear wave velocity data are shown on Table 2.5.4-11. Unit weight data are shown on Table 2.5.4-1. Values for shear modulus are tabulated during analysis with the SHAKE 2000 program **(Bechtel 2000)**, and the low strain values are also shown on Tables 2.5.4-2 for the existing soils and rock, and on Table 2.5.4-10 for the compacted backfill.

From EPRI **(EPRI TR-102293 1993)**, the dynamic shear modulus reduction is derived in terms of depth for granular soils (Upper and Lower Sand Strata) and in terms of Plasticity Index (PI) for cohesive soils (Blue Bluff Marl).

The EPRI curves for sands **(EPRI TR-102293 1993, Figure 7.A-18)** were used to derive the shear modulus reduction factors for the granular soil strata (compacted backfill and Lower Sand Stratum). The EPRI curves for clays **(EPRI TR-102293 1993, Figure 7.A-16)** were used to derive the shear modulus reduction factors for the Lisbon Formation using PI = 25 percent. The shear modulus reduction factors are provided in Table 2.5.4-12 and Figure 2.5.4-9.

The shear modulus reduction factors developed for the neighboring SRS and contained in Lee (1996) were also used in the analysis. The SRS curves were selected based on their stratigraphic relationship to the Voglte 3 and 4 site. The SRS curve labeled as Blue Bluff Marl in Table 2.5.4-13 and on Figure 2.5.4-10 is based on the Dry Branch Formation and the Santee Formation, the SRS stratigraphic equivalent to the Vogtle Blue Bluff Marl. Degradation curves for the compacted backfill were not developed for SRS. The mean site reduction site amplification factors using EPRI and SRS shear modulus degradation relationships were weighted equally as described in Section 2.5.2.5.1.2.1.

2.5.4.7.2.1.2 COL Analysis

Site-specific dynamic shear modulus reduction curves were developed from RCTS test results on samples from the Blue Bluff Marl and Lower Sand strata as well as proposed borrow materials for the compacted backfill, taken during the COL investigation. Index testing was also conducted on these samples. Results of index and RCTS testing are included in Attachment G of Appendix 2.5C.

In the Blue Bluff Marl, four relatively undisturbed samples (Pitcher samples) were tested. Two samples disclosed low plasticity indices (PI =26 and 27) while two disclosed high PI values (46 and 69). The shear modulus reduction data was plotted against shearing strain and overlain on the EPRI curves for clay **(EPRI TR-102293 1993, Figure 7.A-16)**. The site specific data followed trends consistent with the EPRI relationships for PI. Site specific curves were derived for low PI material and high PI material based on the similarity of the EPRI PI curves.

In the Lower Sand Stratum, five relatively undisturbed samples (Pitcher samples) were tested. Three were identified as sand and two were identified as low plasticity clays. The shear modulus reduction data were plotted against shearing strain and overlain on the EPRI curves for depth for granular soils **(EPRI TR-102293 1993, Figure 7.A-18)**. Note that RCTS data for the clayey samples were evaluated against the EPRI curves for clay; however, the damping relationships disclosed in these tests (as discussed later) were not consistent with the EPRI clay relationships. The site specific data followed trends consistent with the EPRI relationships for depth for granular soils. Site specific curves were derived for the sand and the clay materials in the Lower Sand stratum based on the similarity of the EPRI depth curves.

Five bulk samples from test pits in proposed borrow sources were identified for testing. Moisture-density (ASTM D 1557) and index testing were conducted on these samples. The fines content of these samples ranged from about 8 to 25 percent. RCTS tests were conducted on each bulk sample (using the same loading schedule) at two different levels of compaction (95% and 97% or 95% and 100%). The assigned confining pressures for the RCTS testing were determined based on representative depths throughout the proposed 90-ft column of backfill. Test results disclosed little variation based on the level of compaction. The shear modulus reduction data was plotted against shearing strain and overlaid on the EPRI curves for depth for granular soils **(EPRI TR-102293 1993, Figure 7.A-18)**. Test results for samples at low confining pressures disclosed similar trends, as did test results for samples at higher confining pressures. The site specific data followed trends consistent with the EPRI relationships for depth for granular soils. Site specific damping curves for borrow material were developed for samples under low confining pressure (depths less than 25 ft) and for samples under higher confining pressures (greater than 25 ft) based on the similarity of the EPRI curves for depth for granular soils.

Site specific shear modulus reduction curves developed from the RCTS testing of COL samples are provided in Table 2.5.4-12a and Figure 2.5.4-9a. These data were used to evaluate the site response as described in Section 2.5.2.9.3.

2.5.4.7.2.2 Damping

2.5.4.7.2.2.1 ESP Analysis

The publications cited above address the variation of soil damping with cyclic shear strain as well as the variation of shear modulus with shear strain.

From EPRI **(EPRI TR-102293 1993)**, the damping ratio is derived in terms of depth for granular soils (Upper and Lower Sand Strata) and in terms of PI for cohesive soils (Blue Bluff Marl).

The EPRI curves for sands **(EPRI TR-102293 1993, Figure 7.A-19)** were used to derive the damping ratios for the granular soil strata (compacted backfill and Lower Sand Stratum). The EPRI curves for clays **(EPRI TR-102293 1993, Figure 7.A-17)** were used to derive the damping ratios for the Lisbon Formation using PI = 25 percent. The damping ratios are provided in Table 2.5.4-12 and Figure 2.5.4-11.

The damping ratio values developed for the neighboring SRS and contained in Lee (1996) were also used in the analysis. The SRS curves were selected based on their stratigraphic relationship to the Voglte 3 and 4 site. The SRS curve labeled as Blue Bluff Marl in Table 2.5.4-13 and on Figure 2.5.4-12 is based on the Dry Branch Formation and the Santee Formation, the SRS stratigraphic equivalent to the Vogtle Blue Bluff Marl. Degradation curves for the compacted backfill were not developed for SRS. The mean site reduction site amplification factors using EPRI and SRS shear modulus degradation relationships were weighted equally as described in Section 2.5.2.5.1.2.1.

2.5.4.7.2.2.2 COL Analysis

Site-specific damping curves were developed from RCTS test results on samples from the Blue Bluff Marl and Lower Sand strata as well as proposed borrow materials for the compacted backfill, as similarly described in Section 2.5.4.7.2.1.2.

The RCTS damping relationships for the Blue Bluff Marl samples were plotted and overlain on the EPRI curves for clay **(EPRI TR-102293 1993, Figure 7.A-17)**. The site specific data followed trends consistent with the EPRI damping relationships for PI. Site specific curves were derived for low PI material and high PI material based on the similarity of the EPRI PI curves.

The RCTS damping relationships for the Lower Sand Stratum samples were plotted against shearing strain and overlain on the EPRI curves for depth for granular soils **(EPRI TR-102293 1993, Figure 7.A-19)**. The damping relationships for the clayey samples were evaluated against EPRI curves for clay; however, these data disclosed lower damping values at lower shear strains. Instead the RCTS data were more closely aligned with the EPRI relationships with depth for granular soils. The site specific data for both sand and clay samples followed trends consistent with the EPRI relationships for depth for granular soils. Therefore, site specific damping curves were derived for the sand and the clay materials in the Lower Sand stratum based on the similarity of the EPRI curves for depth for granular soils.

The RCTS damping relationships for the proposed borrow sources were plotted against shearing strain and overlain on the EPRI curves for depth for granular soils (EPRI TR-102293 1993, Figure 7.A-19). Test results for samples at low confining pressures disclosed similar trends, as did test results for samples at higher confining pressures. The site specific data followed trends consistent with the EPRI relationships for depth for granular soils. Site specific damping curves for borrow material were developed for samples under low confining pressure (depths less than 25 ft) and for samples under higher confining pressures (greater than 25 ft) based on the similarity of the EPRI curves for depth for granular soils.

Site specific damping curves developed from the RCTS testing of COL samples are provided in Table 2.5.4-12a and Figure 2.5.4-11a. These data were used to evaluate the site response as described in Section 2.5.2.9.3.

After randomization, the damping curves were cut off at 15 percent damping ratio per NUREG-0800, Section 3.7.2 (1996).

2.5.4.7.3 Soil/Rock Column Amplification/Attenuation Analysis

The SHAKE2000 **(Bechtel 2000)** computer program was used to compute the site dynamic responses for the soil/rock profiles described in Section 2.5.4.7.1. The computation was performed in the frequency domain using the complex response method. Section 2.5.2.5 describes in detail the soil/rock column amplification/attenuation analysis based on the ESP soil column.

SHAKE2000 uses an equivalent linear procedure to account for the non-linearity of the soil by employing an iterative procedure to obtain values for shear modulus and damping that are compatible with the equivalent uniform strain induced in each sublayer. At the outset of the analysis, a set of properties (based on the values of shear modulus and damping presented in Section 2.5.4.7.1, and total unit weight) was assigned to each sublayer of the soil profile. The analysis was conducted using these properties, and the shear strain induced in each sublayer was calculated. The shear modulus and damping ratio for each sublayer was then modified based on the shear modulus and damping ratio versus strain relationships presented in Section 2.5.4.7.2. The analysis was repeated until strain-compatible modulus and damping values were achieved.

2.5.4.7.4 Two-Dimensional Effects Site Response Analysis (Bathtub Model)

The model for the site dynamic response analysis as described in Section 2.5.2.5 depicted the backfill above the Blue Bluff Marl as a continuum. The model did not account for the extent of the excavation and backfill and any impacts the Upper Sands have on the site response. These impacts were evaluated by considering the site response with the Upper Sands in place and with these materials replaced with backfill. The average shear wave profile of the Upper Sands as developed from the COL data, as shown on Figure 2.5.4.6a, was used to characterize shear wave velocity of the Upper Sands. A discussion of this analysis and results are presented in Section 2.5.2.9.2.

2.5.4.7.5 Comparison of ESP vs. COL Soil Column

Subsurface data was collected and evaluated at the site during two distinct phases referred to as the ESP investigation and COL investigation (including the Phase 1 test pad program) as presented in Section 2.5.4.3. The ESP investigation was limited in scope and broad in aerial coverage; whereas the COL investigation was more focused in coverage (to the power block area) and extensive in scope. Subsurface data, including shear wave velocity, from the ESP investigation were taken from widely spaced borings. One of these boreholes (B-1003) extended through the entire soil column (over 1,000 ft) and into the underlying sedimentary rock of the Triassic Basin. Subsurface data from VEGP Units 1 and 2 and other regional sources were also evaluated. Soil non-linearity curves obtained from EPRI and the nearby Savannah River Site (SRS) were assigned based on soil type and depth. The resulting ESP soil column was used in the amplification/attenuation analysis in Section 2.5.2.5.

The COL investigation provided numerous additional subsurface data specific to the powerblock areas of Units 3 and 4. The COL investigation was taken to exploration depths of 420 ft. ESP data taken within the powerblock areas were compiled with the COL data to develop the COL soil column. These data included averaged shear wave velocities, averaged strata thicknesses and densities. A thick clay layer (approximately 70 ft) encountered in the Lower Sands, as discussed in Section 2.5.4.2.2.3, was incorporated into the COL soil column as shown on Figure 2.5.4-7a. Site specific soil non-linearity curves for the various strata, including the clay soils in the Lower Sands, were developed from RCTS testing of representative COL samples and are included in the COL soil column. These data were discussed in Section 2.5.4.7.2 and are presented in Figures 2.5.4-9a (G/Gmax curves) and 2.5.4-11a (damping curves). Site specific dynamic properties of the compacted backfill were developed during the COL laboratory testing program and the Phase 1 test pad program and are included in the COL soil column.

The stratification and shear wave velocity profiles for the ESP and COL soil columns are presented in Figure 2.5.4-7a. The offset in soil stratification between the soil columns reflects refinements due to the additional data collected during the COL investigation. The stratification of the ESP soil column is based on the deep boring, B-1003. The stratification of the COL soil column is based on numerous additional borings in the power block areas. The data disclosed thicker near surface strata as compared boring B-1003. No additional stratification or shear wave velocity data below the top of the Snapp Formation in the Lower Sands were collected during COL investigation; therefore, the COL soil column stratification and shear wave velocity profiles between the Snapp Formation and the top of the Triassic Basin bedrock were carried over from the ESP soil column with the same strata thicknesses but slightly shifted in depth to match the thicker near surface strata. Comparison of the two shear wave velocity profiles indicates good agreement between the data sets. Trends within the strata are consistent.

Comparisons of the soil non-linearity curves used for ESP and COL are presented in the attached Figures 2.5.4-19a through 2.5.4-20c. Figures 2.5.4-19a, 19b, and 19c illustrate the normalized shear modulus vs. shear strain curves for compacted backfill, Blue Bluff Marl, and Lower Sands, respectively. Figures 2.5.4-20a, 20b, and 20c illustrate the soil damping vs. shear strain curves for the same strata. The figures include both the site specific curves developed during the COL investigation and the EPRI and SRS model curves assigned during the ESP investigation. The COL site specific data for the Lower Sands includes non-linearity curves for both sand and clay materials in this stratum. Generally the figures suggest that the subsurface soils behave more linearly (provides a smaller reduction in shear modulus and less damping) than the models used for the ESP investigation.

The COL soil column, including shear wave velocity and site specific non-linearity relationships as described here was used in the site response sensitivity analysis to evaluate the effects of the COL data to the ESP data as described in Section 2.5.2.9.3.

Table 2.5.4-12a Summary of Modulus Reduction and Damping Ratio Values - Site Specific

Stratum	Backfill				Blue Bluff Marl				Lower Sands			
Sub strata	<25ft		≥25ft		Low PI		High PI		Sands		Clay (Congaree/ Snapp)	
Shear Strain (%)	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio
0.00010	1	0.97	1	0.62	1	1.44	1	1	1	0.62	1	0.86
0.00032	1	1.05	1	0.62	1	1.56	1	1.05	1	0.62	1	0.87
0.00100	0.998	1.05	1	0.7	1	1.67	1	1.32	1	0.7	1	0.93
0.00359	0.942	1.44	0.975	0.89	0.96	2.34	0.9965	1.71	0.997	0.89	0.99	1.21
0.01019	0.826	2.26	0.902	1.3	0.867	3.23	0.97	2.3	0.954	1.32	0.928	1.8
0.03170	0.603	4.55	0.748	2.6	0.673	5.75	0.88	3.97	0.858	2.6	0.8	3.62
0.10000	0.355	8.97	0.495	5.64	0.395	10.63	0.679	6.715	0.649	5.59	0.56	7.54
0.30690	0.172	14.94	0.269	10.65	0.187	16.39	0.433	11.115	0.411	10.65	0.327	13
0.65313	0.089	19.38	0.158	14.73	0.1	19.08	0.2785	14.545	0.263	14.68	0.198	17.42
1.00000	0.072	22.12	0.117	17.11	0.068	19.12	0.217	15.77	0.209	17.11	0.154	19.87

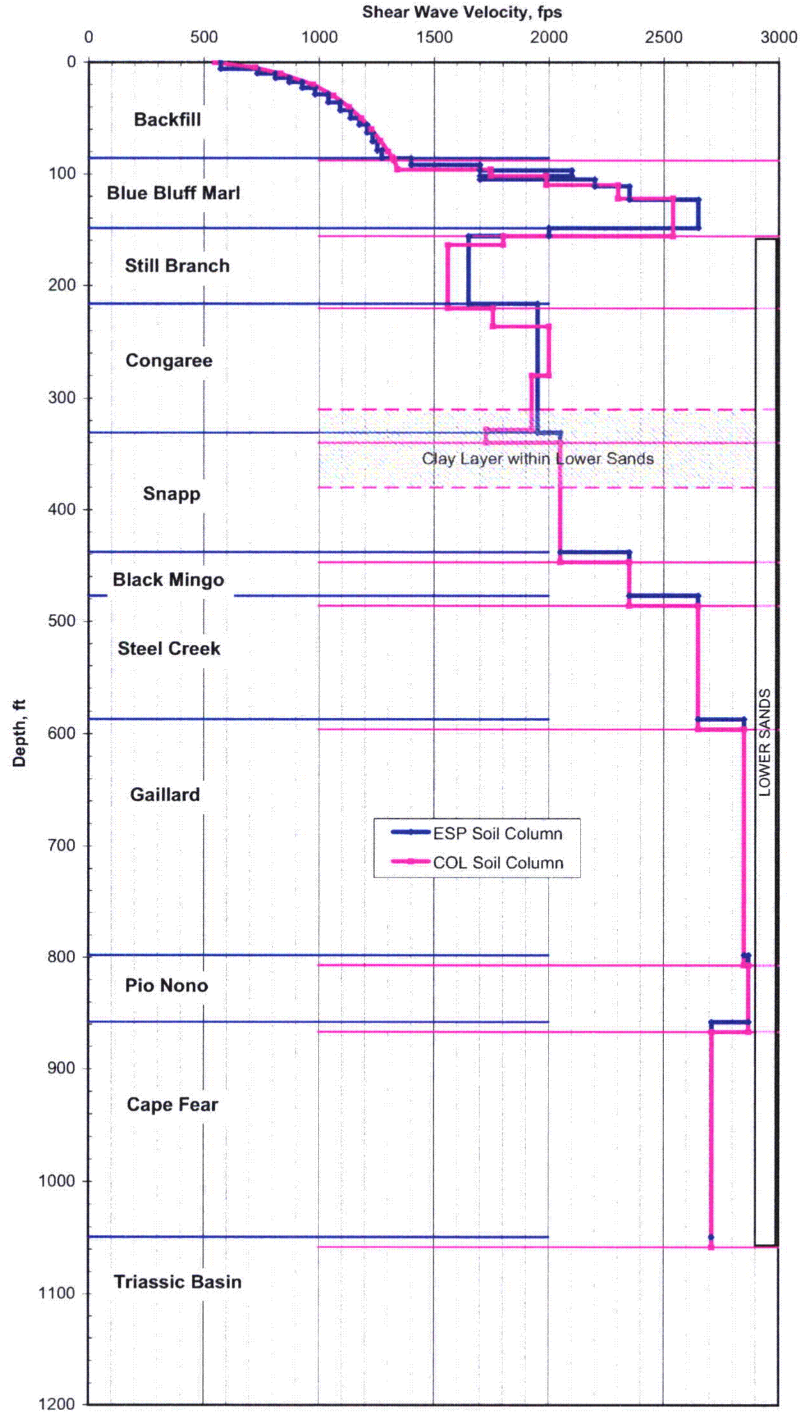

Figure 2.5.4-7a Shear Wave Velocity Profile – ESP and COL Soil Columns

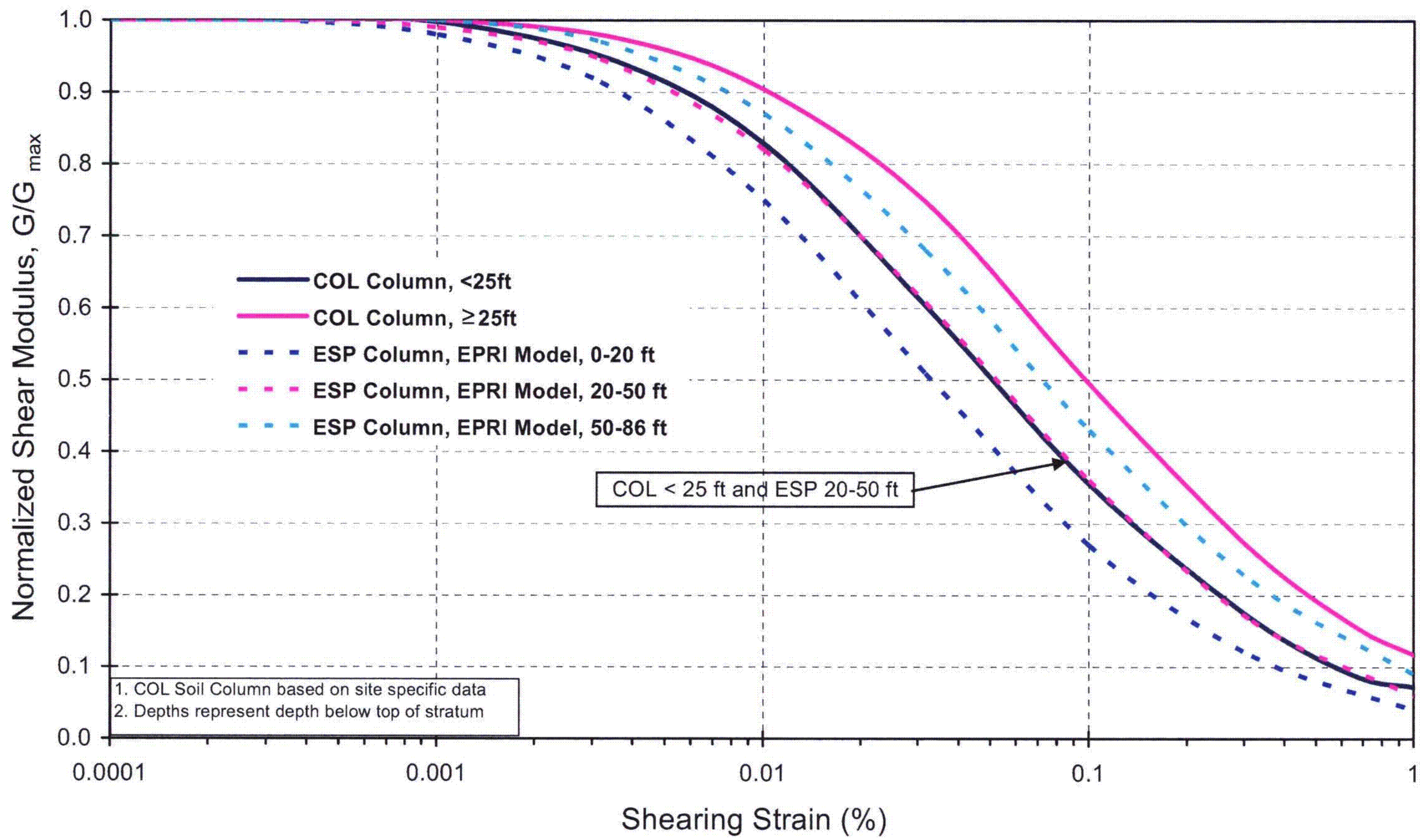

Figure 2.5.4-19a Comparison of Shear Modulus Reduction Curves - Backfill Soils

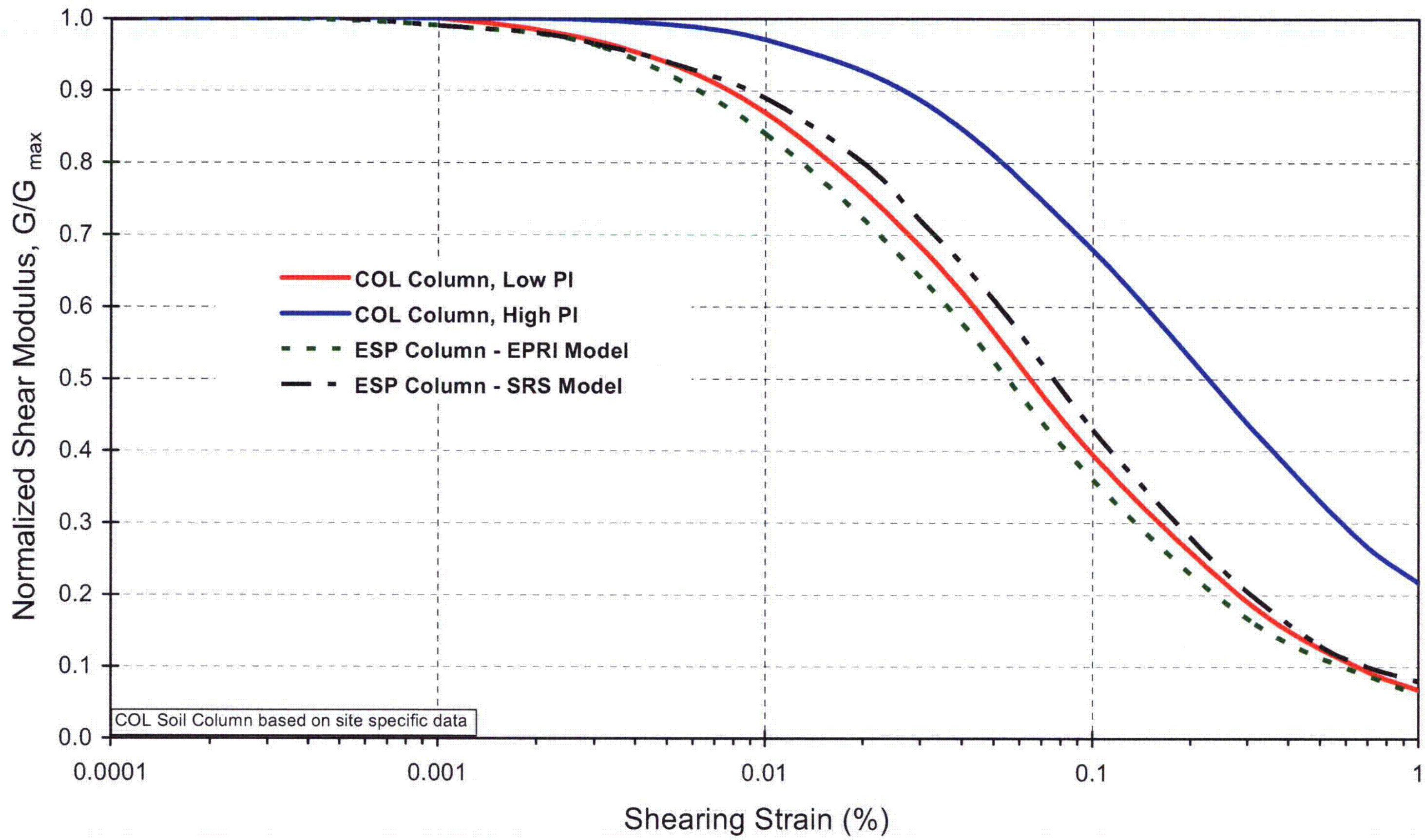

Figure 2.5.4-19b Comparison of Shear Modulus Reduction Curves - Blue Bluff Marl

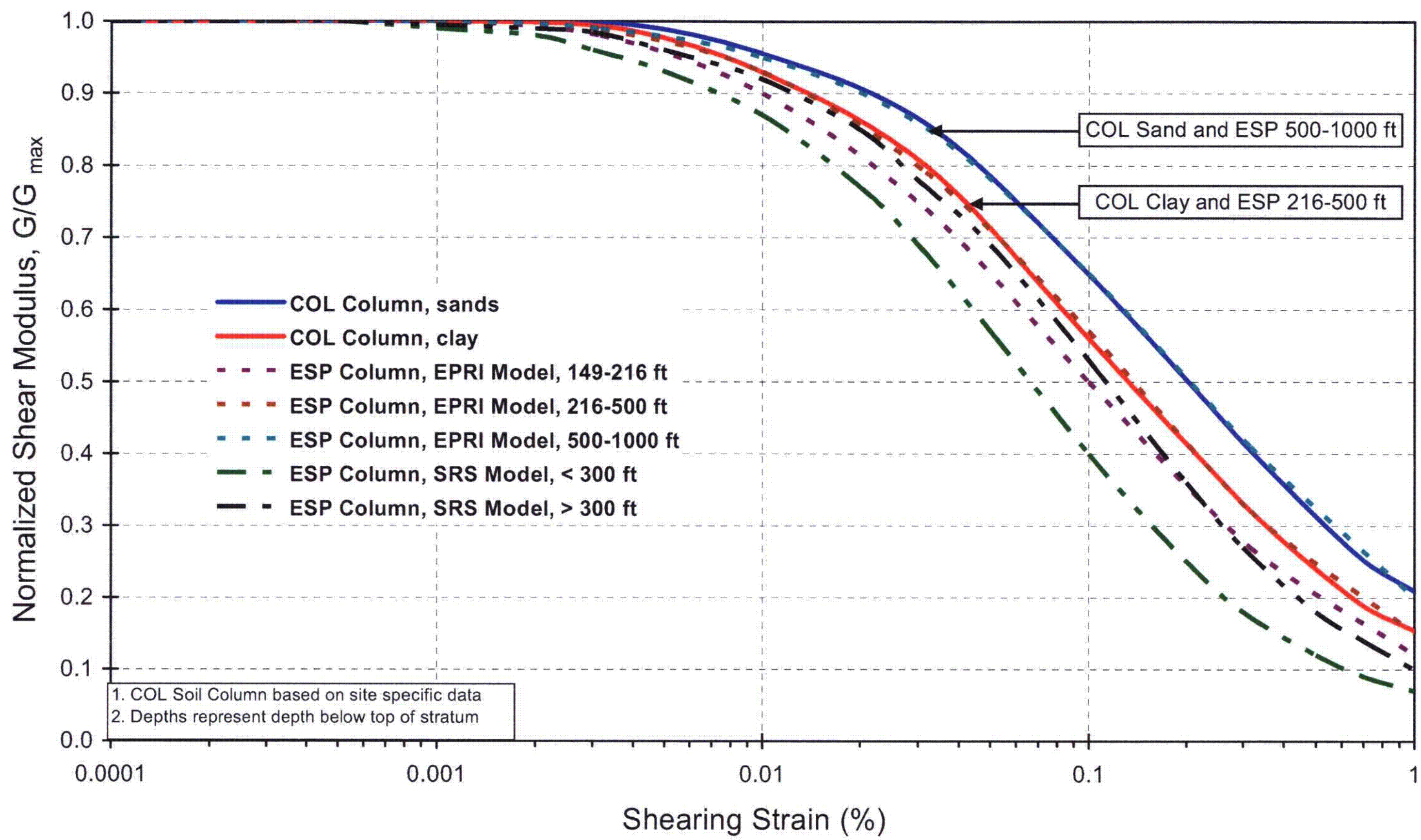

Figure 2.5.4-19c Comparison of Shear Modulus Reduction Curves - Lower Sands

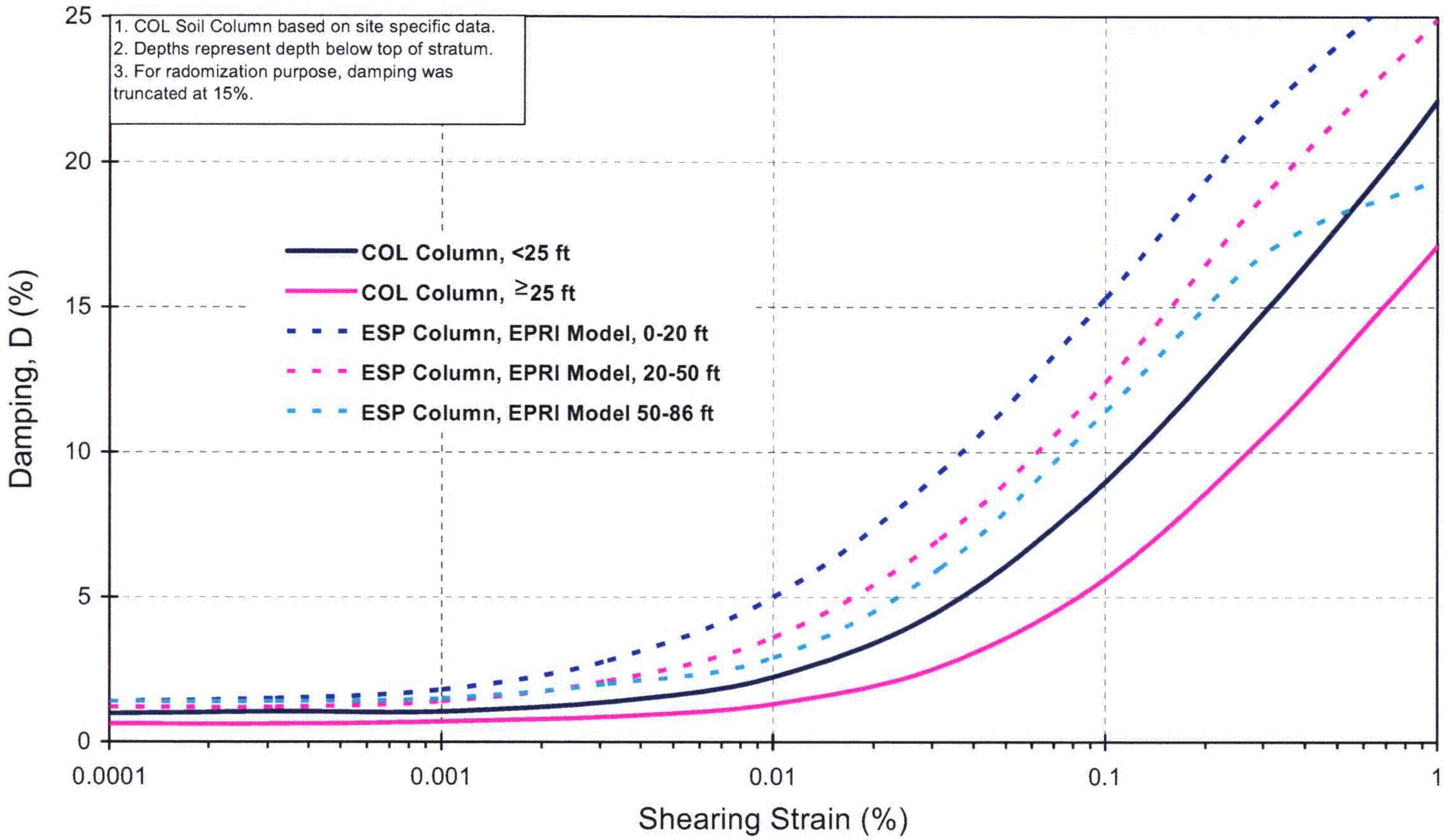

Figure 2.5.4-20a Comparison of Damping Curves - Backfill Soils

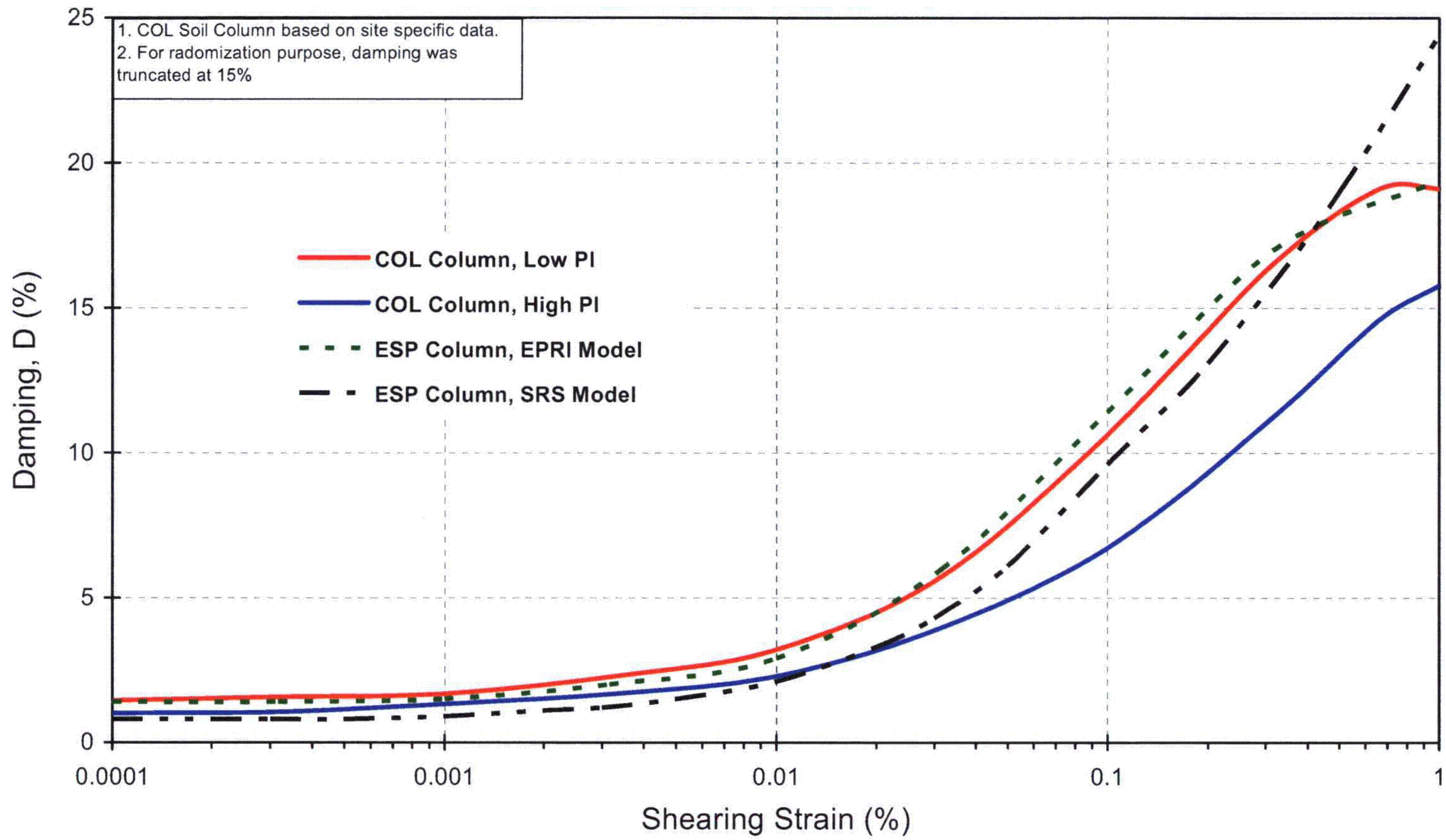

Figure 2.5.4-20b Comparison of Damping Curves - Blue Bluff Marl

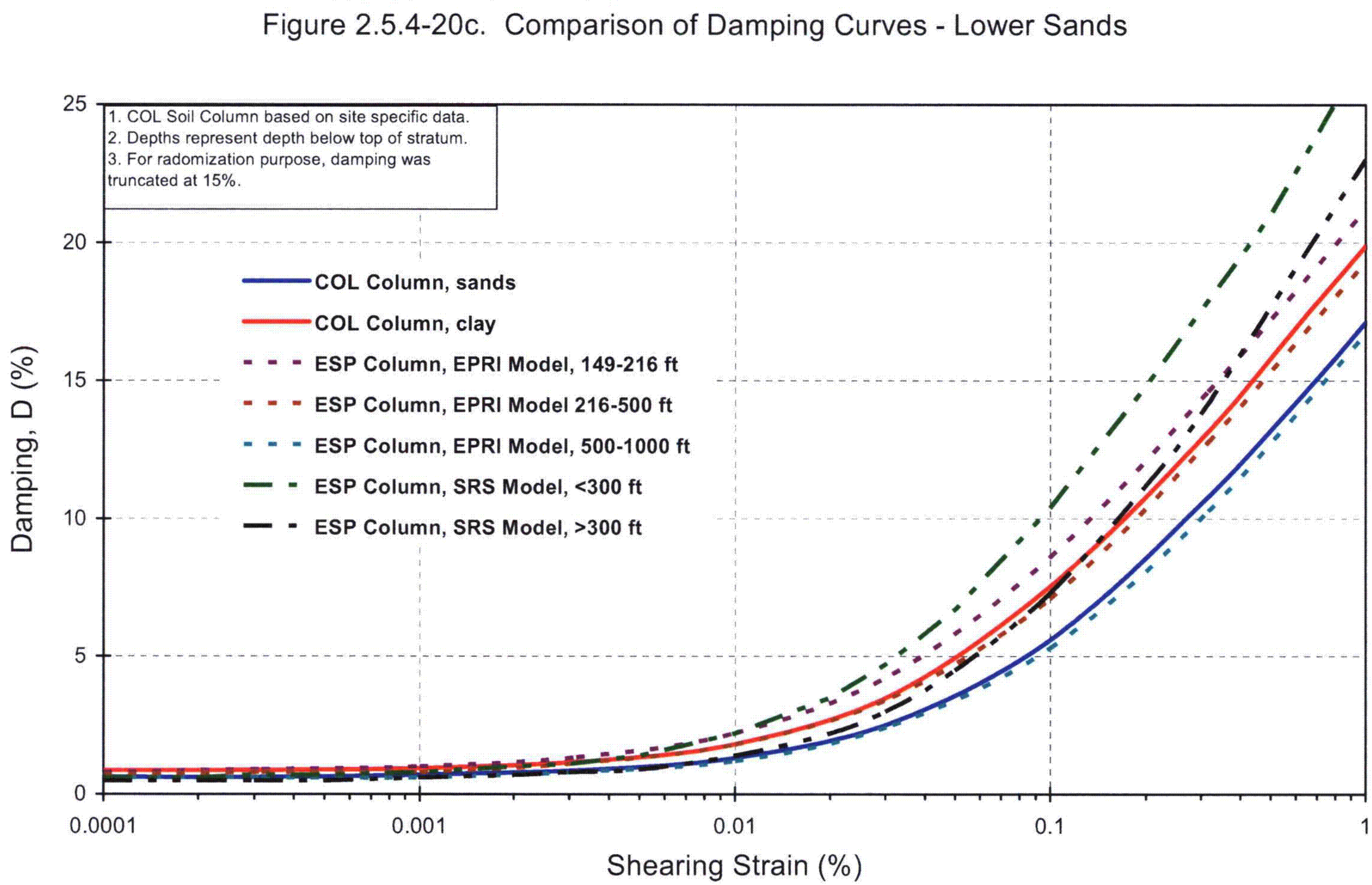

Figure 2.5.4-20c Comparison of Damping Curves - Lower Sands