Chapter 12

GEOTECHNICAL EARTHQUAKE ENGINEERING

FINAL

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

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CHAPTER 12

GEOTECHNICAL EARTHQUAKE ENGINEERING

12.1 INTRODUCTION

Geotechnical earthquake engineering consists of evaluating the earthquake hazard and the effects of the hazard on the transportation structure being designed. This is accomplished by characterizing the subsurface soils, determining the earthquake hazard, evaluating the local site effects on the response spectra, and developing an acceleration design response spectrum (ADRS) for use in designing bridges and other transportation structures.

SCDOT has made a commitment to design transportation systems in South Carolina so as to minimize their susceptibility to damage from earthquakes. The SCDOT *Seismic Design Specifications for Highway Bridges* establishes the seismic design requirements for the design of bridges in the South Carolina highway transportation system. This chapter presents geotechnical earthquake engineering design requirements for evaluating ground shaking using either SC Seismic Hazard maps or by performing a site-specific response analysis. The SC Seismic Hazard Maps and Deaggregation Charts are discussed in Chapter 11. Geotechnical seismic analysis and design guidelines for evaluating soil liquefaction potential, analyzing liquefaction induced hazards, seismic slope stability, and analyzing seismic lateral loadings are contained in Chapters 13 and 14.

The GDS performs the following types of geotechnical earthquake engineering analyses:

1. Geotechnical Seismic Site Characterization (Chapter 12)
2. Performs Earthquake Hazard Analyses – Liquefaction, etc. (Chapter 13)
3. Generates Earthquake Ground Motions - Time Histories (Chapter 11)
4. Determines Earthquake Design Parameters – PGA, PSA, M_w, etc. (Chapter 11)
5. Develops Acceleration Design Response Spectrum (ADRS) curves (Chapter 12)
6. Develops Geotechnical Earthquake Engineering Design Guidelines (Chapter 14)
7. Reviews Consultant Geotechnical Earthquake Engineering Reports (Chapter 3)

12.2 GEOTECHNICAL EARTHQUAKE ENGINEERING DESIGN

The geotechnical earthquake engineering requirements for determining the seismic hazard and associated response have been developed for the design of “Typical SCDOT Bridges” as defined by Sections 1.4 and 1.5 of the SCDOT *Seismic Design Specifications for Highway Bridges*. Bridges not meeting the definition of “Typical SCDOT Bridges” include suspension bridges, cable-stayed bridges, arch type bridges, movable bridges, and bridge spans exceeding 300 feet. For these non-typical bridges, the PCS/GDS will specify and/or approve appropriate geotechnical earthquake engineering provisions on a project specific basis. The geotechnical earthquake engineering requirements in this Manual also apply to the design of geotechnical roadway structures such as roadway embankments, earth-retaining systems, and other miscellaneous transportation related structures.

The preliminary geotechnical engineering report (PGER) typically contains a geotechnical earthquake hazard analysis that includes the determination of a Site Class based on available subsurface information and a horizontal acceleration design response spectrum (ADRS) to be used for preliminary design of the bridge structure. The final geotechnical engineering report (BGER or RGER) contains the results of the final geotechnical subsurface investigation and modifies, if necessary, the Site Class and the horizontal acceleration design response spectrum (ADRS) curves.

12.3 DYNAMIC SOIL PROPERTIES

12.3.1 Soil Properties

A project specific subsurface geotechnical investigation is typically required in accordance with the subsurface investigation guidelines provided in Chapter 4. Basic soil properties will be obtained in accordance with the field and laboratory testing procedures specified in Chapter 5. Basic soil properties can be directly measured by field and laboratory testing results or can be correlated from those results as described in Chapter 7. Dynamic soil properties such as shear wave velocity, V_S, should be measured in the field (Chapter 5) and correlated as indicated in this Chapter when insufficient field measurements are available. Other dynamic properties such as shear modulus curves, equivalent viscous damping ratio curves, and residual strength of liquefied soils are determined as indicated in this Chapter.

12.3.2 Soil Stiffness

One of the required soil properties needed to perform a soil response analysis is the soil stiffness. Soil stiffness is characterized by either small-strain shear-wave velocity or small-strain shear modulus. The small-strain shear wave velocity, V_S, is related to small-strain shear modulus, G_{max}, by the following equation.

$$\boldsymbol{G}_{\max} = \rho \boldsymbol{V_S}^2 = \frac{\gamma_T \boldsymbol{V_S}^2}{\boldsymbol{g}} \qquad \text{Equation 12-1}$$

Where the mass density of soil, ρ, is equal to the total unit weight, γ_T, of the soil divided by the acceleration of gravity (g = 32.174 ft/sec^2 = 9.81 m/sec^2).

Typical values of small-strain shear wave velocity, V_S, and small-strain shear modulus, G_{max}, for various soil types are shown in Table 12-1. Additional guidance on selecting appropriate shear wave velocities can be obtained by reviewing the database range of shear wave velocities for different South Carolina soil deposits indicated in Tables 12-3 and 12-9. Typical small-strain shear wave velocity profiles for different parts of South Carolina are provided in Section 12.3.3.

Table 12-1, Typical Small-Strain Shear Wave Velocity and Initial Shear Modulus
(Based on Hunt, 1984 and Kavazanjian, 1998)

Soil Type	**Mass Density, ρ**	**Total Unit Weight, γ**	**Small-strain Shear Wave Velocity, V_S**		**Initial Shear Modulus, G_{max}**	
	kg/m^3	**pcf**	**m/s**	**ft/s**	**kPa**	**psi**
Soft Clay	1,600	100	40 – 90	130 – 300	2,600 – 13,000	400 – 2,000
Stiff Clay	1,680	105	65 – 140	210 – 500	7,000 – 33,000	1000 – 5,700
Loose Sand	1,680	105	130 – 280	420 – 920	28,400 – 131,700	4,000 – 19,200
Dense Sand and Gravel	1,760	110	200 - 410	650 – 1,350	70,400 – 300,000	10,000 – 43,300
Residual Soil (PWR, IGM)	2,000	125	300 - 600	1,000 – 2,000	180,000 – 720,000	27,000 – 108,000
Piedmont Metamorphic and Igneous Rock (Highly – Moderately Weathered)	2,500	155	760 – 3,000	2,500 – 10,000	1,400,00 – 22,500,000	209,000 – 3,400,000
0 <RQD < 50 RQD = 65 [1] RQD = 80 [1] RQD = 90 [1] RQD = 100 [1]			600 760 1,500 2,500 3,400	2,000 2,500 5,000 8,000 11,000		
Basement Rock (Moderately Weathered to Intact)	2,600	165	> 3,400	> 11,000	> 30,000	> 4,300,000

[1] Typical Values, Linear interpolate between RQD values

When performing a geotechnical subsurface investigation it is typically preferred to measure site-specific small-strain shear wave velocity, V_S, as described in Chapters 4 and 5. When site-specific shear wave velocities, V_S, are not available or needs to be supplemented, an estimation of the shear wave velocity, V_S, can be made by the use of correlations with in-situ testing such as the Standard Penetration Test (SPT) or the Cone Penetration Test (CPT). Procedures for estimating dynamic properties of soils in South Carolina have been developed by Andrus et al. (2003). The procedures for correlating SPT and CPT results with shear wave velocity, V_S, have been summarized in Sections 12.3.2.1 and 12.3.2.2, respectively. For a more detailed description of the procedures to estimate dynamic properties see Andrus et al. (2003). A review of SPT calculated shear wave velocity relationships reveals that few relationships have been developed for clays. This is likely due to SPT blow counts (N) not being the appropriate test for cohesive soils, particularly since soft clays would have SPT blow counts that would be close to zero.

The SPT correlations for shear wave velocity, V_S, use the standardized SPT blow count, N^*_{60}, that is defined in Chapter 7. The CPT correlations for shear wave velocity, V_S, use the measured CPT tip resistance, q_c, that is defined in Chapter 5.

12.3.2.1 SPT - Shear Wave Velocity, V_S, Estimation of SC Sands

Recommended equations to estimate shear wave velocities, V_S, for South Carolina soils are based on standardized SPT blow count (N^*_{60}), depth (Z), Fines Content (FC), geologic age and location of deposit, and Age Scaling Factor (ASF). Equations for estimating shear wave velocities, V_S, of South Carolina sands are provided in Table 12-2 and shown in Figure 12-1.

Table 12-2, SPT (N^*_{60}) - Shear Wave Velocity, V_S, Equations for SC Sand (Andrus et al., 2003)

Fines Content, *FC*	Equation for Predicting V_S (m/s) [(1)]	Equation No.
< 40%	$V_S = 72.9(N^*_{60})^{0.224} Z^{0.130} ASF$	Equation 12-2
10% to 35%	$V_S = 72.3(N^*_{60})^{0.228} Z^{0.152} ASF$	Equation 12-3
< 10%	$V_S = 66.7(N^*_{60})^{0.248} Z^{0.138} ASF$	Equation 12-4

(1) N^*_{60} = blows/0.3m = blows/ft (Section 7.8.1) and Z = depth in meters, ASF = Age Scaling Factors

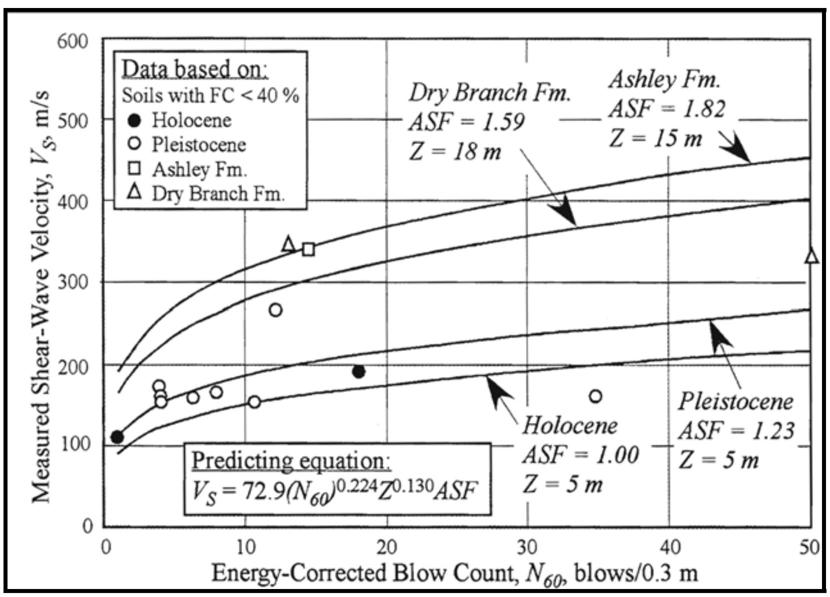


Figure 12-1, SPT (N_{60}) vs. Shear Wave (V_S) (Andrus et al., 2003)

Recommended age scaling factors (ASF) based on Andrus et al. (2003) are provided in Table 12-3.

Table 12-3, Recommended Age Scaling Factors (ASF) for SPT (Andrus et al., 2003)

Geologic Age and Location of Deposits	Fines Content [1], *FC* (%)	Age Scaling Factor, *ASF*	Database Range of Shear Wave Velocity, V_S	
			m/s	ft/s
Holocene SC Coastal Plain	< 40%	1.00	110 – 260	360 - 850
	10% to 35%	1.00	120 - 240	400 - 800
	< 10%	1.00	110 – 260	360 - 850
Pleistocene SC Coastal Plain	< 40%	1.23	150 – 270	500 - 900
	10% to 35%	1.08	160	550
	< 10%	1.28	150 – 270	500 - 900
Tertiary SC Coastal Plain Ashley Formation (Cooper Marl)	< 40%	1.82	340	1,100
	10% to 35%	1.71	340	1,100
Tertiary SC Coastal Plain Dry Branch Formation	< 40%	1.59	330 – 350	1,100 – 1,200
	10% to 35%	1.48	330 - 350	1,100 – 1,200

[1] *FC*= % passing #200 sieve

The procedures for using the V_s correlation equations in Table 12-2 are provided in Table 12-4.

Table 12-4, Procedure for Correlating SPT (N^*_{60}) to Shear Wave Velocity, V_s

Steps	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Determine fines content (FC) for soils at each SPT (N_{meas}) at depth (Z).
3	Compute standardized SPT blow count (N^*_{60}) to account for energy variations in SPT equipment. (Section 7.8.1)
4	Calculate shear wave velocity, V_S, for each (N^*_{60}) using Equation 12-2 and the appropriate *ASF* in Table 12-3. Equation 12-2 is the general equation used to estimate V_S for Sands with less than 40% fines content. If the fines content, *FC*, is known more definitive, then a better estimation can be made with Equations 12-3 and 12-4.
5	Plot a profile of calculated shear wave velocities, V_S, with respect to depth. If field shear wave velocity measurements have been made, plot this data on the profile and compare calculated shear wave results, V_S, with the measured V_S to verify appropriateness (accuracy) of SPT-V_S Equations.

12.3.2.2 CPT - Shear Wave Velocity, V_S, Estimation of SC Soils

Recommended equations to estimate shear wave velocities, V_S, for South Carolina soils are based on CPT tip resistance (q_c), depth (Z), soil behavior type (I_c), geologic age and location of deposit, and Age Scaling Factor (*ASF*). Equations for estimating shear wave velocities, V_S, of South Carolina soils are provided in Table 12-5. The CPT – V_S relationship for Holocene, Pleistocene, and Tertiary soils are plotted in Figures 12-2, 12-3, and 12-4, respectively.

Table 12-5, CPT (q_c) - Shear Wave Velocity, V_S, Equations for SC Soils (Andrus et al., 2003)

Soil Behavior Type, I_c	Equation for Predicting V_S (m/s) (1)	Equation No.
All Values	$V_S = 4.63 q_c^{0.342} I_c^{0.688} Z^{0.092} ASF$	Equation 12-5
< 2.05	$V_S = 8.27 q_c^{0285} I_c^{0.406} Z^{0.122} ASF$	Equation 12-6
> 2.60	$V_S = 0.208 q_c^{0.654} I_c^{1.910} Z^{-0.108} ASF$	Equation 12-7

(1) I_c = Soil Behavior Type Index (See Table 12-6)
(2) q_c = CPT tip resistance (kPa), Z = depth in meters, and ASF = Age Scaling Factors

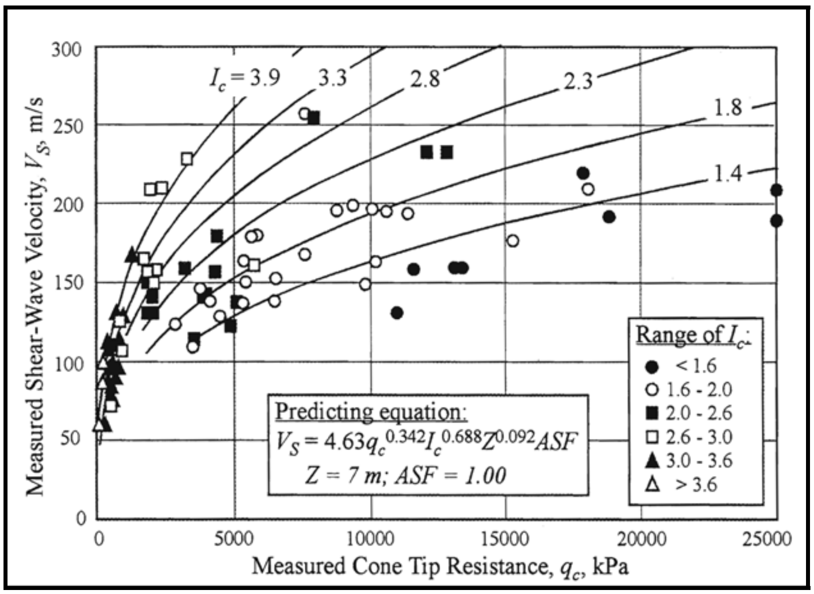


Figure 12-2, CPT – V_S Relationship for Holocene Soils (Andrus et al., 2003)

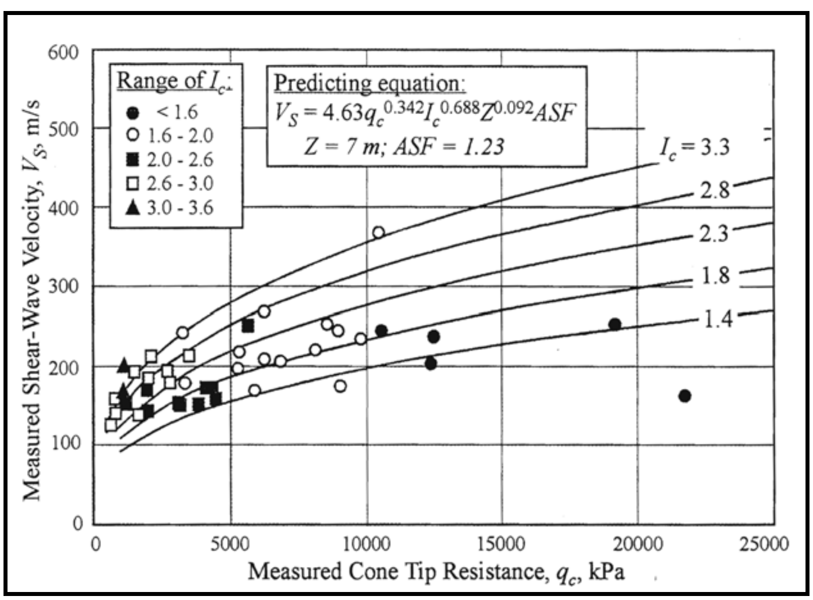


Figure 12-3, CPT – V_S Relationship for Pleistocene Soils (Andrus et al., 2003)

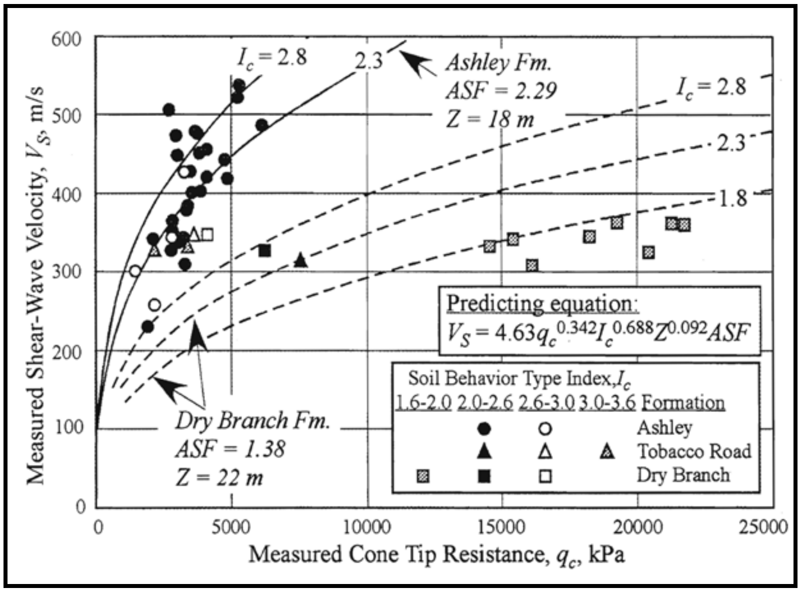


Figure 12-4, CPT – V_S Relationship for Tertiary Soils (Andrus et al., 2003)

Robertson (1990) established general soil behavior type, I_c, values as shown in Figure 12-5.

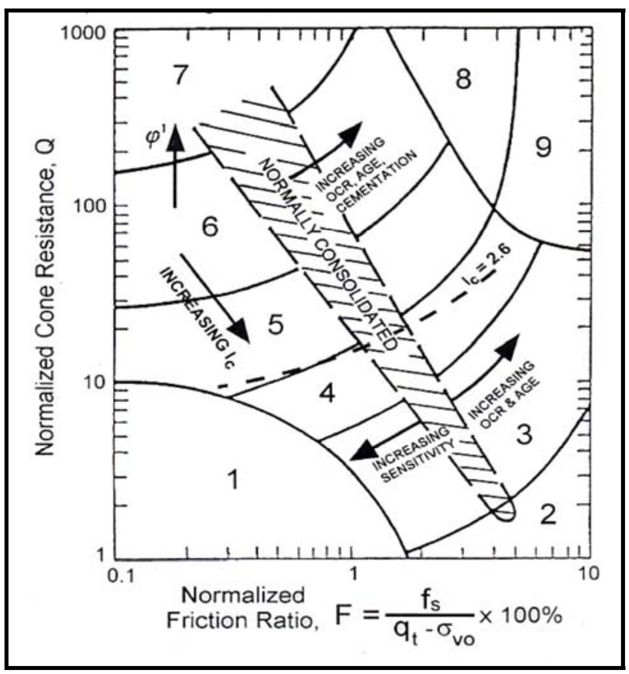


Figure 12-5, Normalized CPT Soil Behavior Type Chart (Robertson, 1990)

Table 12-6 indicates, for soil zones 1 thru 9 (shown in Figure 12-5), the soil behavior type description and the soil behavior index, I_c.

Table 12-6, Soil Behavior Type Index for CPT (Robertson, 1990)

Zone	Soil Behavior Type Description	Soil Behavior Type Index, I_c
1	Sensitive, Fine Grained	---
2	Organic Soils – Peat	$I_c > 3.60$
3	Clays – Silty Clay to Clay	$2.95 < I_c < 3.60$
4	Silt Mixtures – Clayey Silt to Silty Clay	$2.60 < I_c < 2.95$
5	Sand Mixtures – Silty Sand to Sandy Silt	$2.05 < I_c < 2.60$
6	Sands – Clean Sand to Silty Sand	$1.31 < I_c < 2.05$
7	Gravelly Sand to Sand	$I_c < 1.31$
8	Very Stiff Sand to Clayey Sand (1)	---
9	Very Stiff, Fine Grained (1)	---

(1) Heavily overconsolidated or cemented soils

The boundaries between soil zones 2 through 7 shown in Figure 12-5 can be differentiated by a soil behavior index, I_c.

$$I_c = \left[(3.47 - \log Q)^2 + (1.22 + \log F)^2\right]^{0.5}$$ Equation 12-8

The normalized cone resistance, *Q*, and the normalized friction ratio, *F*, are computed using the equations shown in Table 12-7.

Table 12-7, Normalized CPT Q and F Equations (Andrus et al., 2003)

Normalized CPT Value	Equation (1)	Equation No.
Normalized Cone Resistance, *Q*	$Q = \left[\frac{q_c - \sigma'_v}{P_a}\right]\left(\frac{P_a}{\sigma'_v}\right)^n$	Equation 12-9
Normalized Friction Ratio, *F*	$F = \left[\frac{f_s}{q_c - \sigma'_v}\right]100\%$	Equation 12-10

(1) q_c = CPT Tip Resistance (kPa); f_s = CPT Skin Resistance (kPa); P_a = Reference Stress = 100 kPa = 1 atm; σ'_V = Effective Vertical or Overburden Stress (kPa); n = exponent ranging from 0.5 to 1.0 (See Table 12-8)

The soil behavior index, I_c, is computed using Equations 12-8, 12-9, 12-10 and using an iterative procedure developed by Robertson and Wride (1998) as detailed in Table 12-8.

Table 12-8, Soil Behavior Index, I_c, Iterative Computational Procedure (Robertson and Wride, 1998)

1. Calculate soil behavior index, I_c, using n=1.0.
2. If soil behavior index, I_c, is > 2.60, use computed I_c using n=1.0
3. If soil behavior index, I_c, is < 2.60, recalculate I_c using n=0.50
   a. If the recalculated I_c is <2.60, use computed I_c using n=0.50
   b. If the recalculated I_c is >2.60, recalculate I_c using n=0.70

Recommended age scaling factors (*ASF*) based on Andrus et al., (2003) are provided in Table 12-9.

Table 12-9, Recommended Age Scaling Factors (ASF) for CPT (Andrus et al., 2003)

Geologic Age and Location of Deposits	Soil Behavior Description	Soil Behavior Type Index, I_c	Age Scaling Factor, *ASF*	Database Range of Shear Wave Velocity, v_S	
				m/s	ft/s
Holocene SC Coastal Plain	All Soils	All Values	1.00	60 – 260	200 - 850
	Clean Sand Silty Sand	< 2.05	1.00	110 – 260	350 - 850
	Clay, Silty Clayey Silt, Silty Clay	> 2.60	1.00	60 – 230	200 - 750
Pleistocene SC Coastal Plain	All Soils	All Values	1.23	130 – 300	450 – 1,000
	Clean Sand Silty Sand	< 2.05	1.34	160 – 300	500 – 1,000
	Clay, Silty Clayey Silt, Silty Clay	> 2.60	1.16	130 – 250	450 – 1,000
Tertiary SC Coastal Plain Ashley Formation (Cooper Marl)	All Soils	All Values	2.29	230 – 540	750 – 1,800
Tertiary SC Coastal Plain Tobacco Road Formation	All Soils	All Values	1.65	310 – 350	1,000 – 1,150
	Clay, Silty Clayey Silt, Silty Clay	> 2.60	1.42	330 – 350	1,100 – 1,150
Tertiary SC Coastal Plain Dry Branch Formation	All Soils	All Values	1.38	310 – 360	1,000 – 1,200
	Clean Sand, Silty Sand	< 2.05	1.33	310 – 360	1,000 – 1,200

The procedures for using the q_c correlation equations in Table 12-5 are provided in Table 12-10.

Table 12-10, Procedure for Correlating CPT (q_c) to Shear Wave Velocity, V_s

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Calculate soil behavior index, I_c, for soils at each CPT (q_c) at depth (Z) using the Equations 12-8, 12-9, 12-10 and computational procedure listed in Table 12-8.
3	Convert CPT tip resistance, q_c, to kPa and depth, Z, in meters.
4	Calculate shear wave velocity, V_S, for each CPT tip resistance, q_c, value of interest using Equation 12-5 and the appropriate *ASF* value in Table 12-9. Equation 12-5 is the general equation used to estimate V_S for all values of I_c and values of $2.05 \leq I_c \leq 2.60$. A better estimation of shear wave velocity, V_S, can be obtained using Equations 12-6 for $I_c < 2.05$ or Equation 12-7 for $I_c > 2.60$. The *ASF* values in Table 12-9 listed for all values of I_c can be used with the general Equation 12-5. For a better estimation of *ASF*, the *ASF* values associated with soil with $I_c < 2.05$ or for $I_c > 2.60$ can also be used.
5	Plot profile of calculated shear wave velocities, V_S, with respect to depth. If field shear wave velocities measurements have been made, plot this data on the profile and compare calculated shear wave results, V_S, with the measured V_S to verify appropriateness (accuracy) of CPT-V_S Equations.

12.3.2.3 Corrected Shear Wave Velocity, V_{S1}, for Overburden Stress

Some analytical methods require that the shear wave velocity, V_S, be corrected for effects of effective overburden stress, σ'_V. Measured or calculated shear wave velocity, V_S, can be corrected for overburden stress using Equations 12-11 and 12-12.

$$V_{1,S} = V_S C_{VS} = V_S \left(\frac{P_a}{\sigma'_V} \right)^{0.25} \qquad \text{Equation 12-11}$$

$$C_{VS} = \left(\frac{P_a}{\sigma'_V} \right)^{0.25} \leq 1.4 \qquad \text{Equation 12-12}$$

Where effective overburden stress, σ'_V is in kPa and P_a is the reference stress of 100 kPa. The shear wave overburden correction, C_{VS}, is limited to 1.4. The P_a and σ'_V used to compute C_{VS} in Equation 12-12 must be in the same units.

12.3.3 South Carolina Reference Shear Wave Profiles

The shear wave profiles presented in this section are provided for reference purposes only. Project specific shear wave profiles should be developed from shear wave measurements as indicated in Chapter 4 and supplemented to deeper formations by the use of geologic publications, previous investigations, and reference shear wave profiles presented in this section.

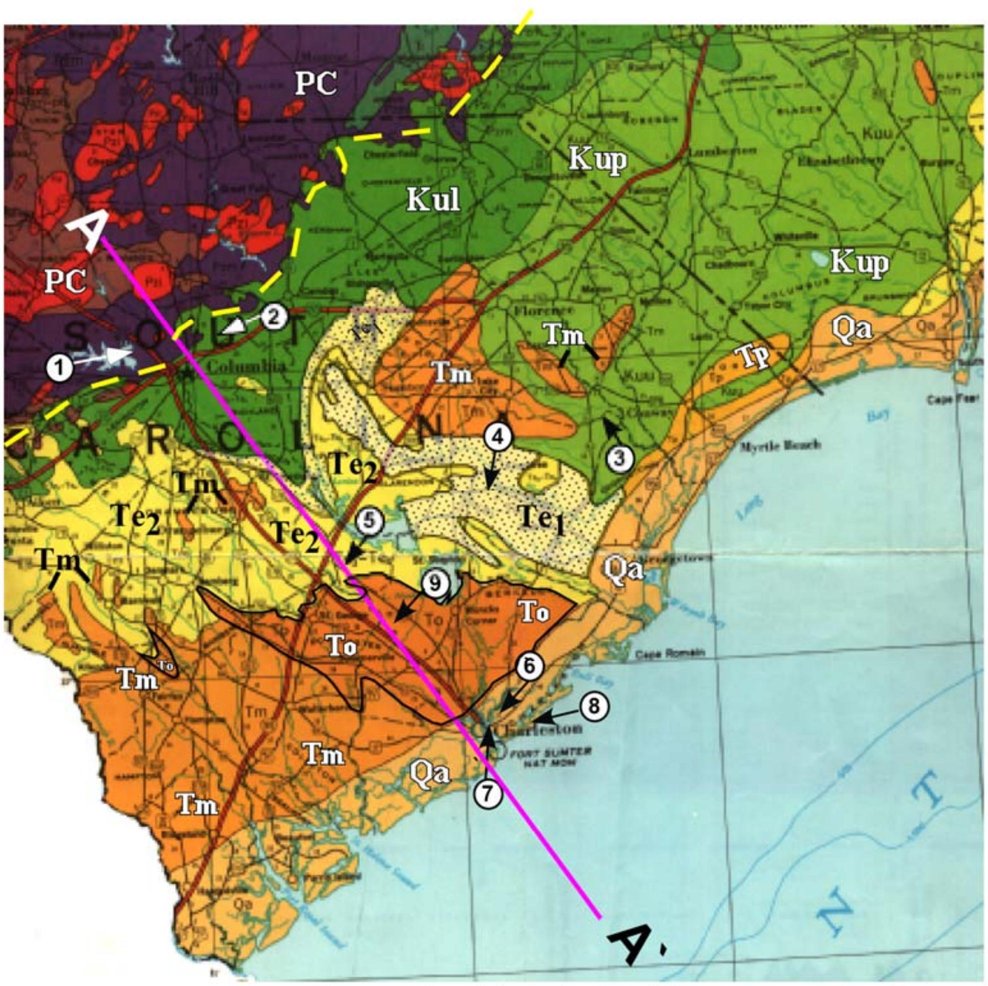
A number of seismic studies have been performed in South Carolina that have yielded shear wave profiles for different parts of the state. The majority of the shear wave profiles in published references are in the Coastal Plain. Shear wave velocities were obtained by one of the following testing methods: Seismic Refraction, Seismic Reflection, Surface Wave (SASW and MASW), Downhole (including Seismic CPT), or Crosshole as described in Chapter 5. When shear wave measurements are not available for soil formations beyond the shear wave testing capabilities, estimates are typically made by using available shear wave data from formations previously tested or by using geologic information.

The shear wave velocity profile information contained in this section has been divided into three sections: USGS Shear Wave Velocity Data, SCEMD Seismic Risk and Vulnerability Study, and Published / SCDOT Shear Wave Velocity Profiles. A brief review of these reference shear wave velocity profiles is presented in the following sections.

12.3.3.1 USGS Shear Wave Velocity Data

The U.S. Geologic Survey (USGS) has compiled shear wave profiles in South Carolina in a report prepared by Odum et al. (2003). Shear wave measurements were obtained by seismic refraction/reflection profiling techniques for nine locations in South Carolina as indicated in Figure 12-6 and listed below:

1. Lake Murray Dam Spillway, Columbia, SC: Paleozoic Rocks of the Carolina Slate Group.
2. Fort Jackson Military Base, Columbia, SC: Cretaceous Tuscaloosa Formation (Middendorf Formation)
3. Deep Creek School: Peedee Formation (Upper Cretaceous)
4. Black Mingo: Black Mingo Formation (lower Eocene-Wilcox Group)
5. Santee Limestone: Santee Limestone (Middle Eocene-Clayborne Group)
6. The Citadel, Charleston, SC: Quaternary deposits (barrier sand facies) overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations) - The Citadel
7. Highway US 17 Overpass next to Ashley River Memorial Bridge: Quaternary deposits overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations)
8. Isle of Palms, Charleston, SC: Quaternary deposits (beach and barrier-island sand facies) overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations)
9. U.S. National Seismograph Network (USNSN) installation site: Quaternary deposits overlying Upper Tertiary Cooper Group (Ashley and Parkers Ferry Formations)



Generalized Geologic Units

Qa	Surface soils and fill, Pleistocene and Holocene of alluvial and marine origin
Tp	Pliocene sandy shell marl
Tm	Miocene sandy phosphatic marl, shaley (Hawthorn Group)
To	Oligocene massive phosphatic marl (Cooper Group-Ashley Fm.)
Te_2	Eocene upper units- calcilutite and limestones (Cooper Gp - Parkers Ferry Fm.) Santee Ls.
	Eocene lower units-clayey to sandy siltstone and limestone (Wilcox Group, Black Mingo Fm.)
Kup	Cretaceous- calcareous clayey sands and limestone (Peedee Fm.)
Kul	Cretaceous- predominantly clean to silty sands and pure clay (Tuscaloosa Fm.)
Pc	Precambrian- metamorphosed early Paleozoic and older sedimentary and volcanic rocks and granitic intrusions (Carolina Slate Group)

Figure 12-6, USGS Nine Study Locations (Odum et al., 2003)

Shear wave (V_S) profiles for the nine USGS sites are summarized in Table 12-11 and shown in Figure 12-7.

Table 12-11, USGS Shear Wave Profile Summary (Odum et al., 2003)

Site No.	Site Name	Latitude (degrees)	Longitude (degrees)	Surficial Geology [1]	Highest V_S in Upper 164' (50 m) (m/s)	Highest V_S in Upper 164' (50 m) (ft/sec)	Description [1]
1	Lake Murray Spillway	35.052	81.210	Fill, Pz	2,674 @ 23 m	8,770 @ 75 ft	Carolina Slate Group (P_z)
2	Fort Jackson	34.028	90.912	K_u	866 @ 27 m	2,840 @ 89 ft	Tuscaloosa Fm
3	Deep Creek School	33.699	79.351	Q?, K_u	710 @ 22 m	2,330 @ 72 ft	Q over Peedee Fm
4	Black Mingo	33.551	79.933	Q, T_l	855 @ 9 m	2,805 @ 30 ft	Q over Eocene Wilcox Group
5	Santee Ls	33.235	80.433	T_l	932 @ 7 m	3,057 @ 23 ft	Santee Limestone
6	The Citadel, Charleston	32.798	79.958	Q, T_u	795 @ 78 m	2,608 @ 256 ft	Q over T_u (Cooper Group)
7	US Hwy. 17, Charleston	32.785	79.955	Fill, Q	247 @ 11 m	810 @ 36 ft	Q over T_u (Cooper Group)
8	Isle of Palms	32.795	79.775	Q_h, T_u	497 @ 23 m	1,630 @ 75 ft	Q over T_u (Cooper Group)
9	USNSN	33.106	80.178	Q, T_u	792 @ 10 m	2,598 @ 33 ft	Q over T_u (Cooper Group)

[1] Definitions: Q – Quartenary; T_u – upper Tertiary; T_l – lower Tertiary; K_u – upper Cretaceous; P_z - Paleozoic

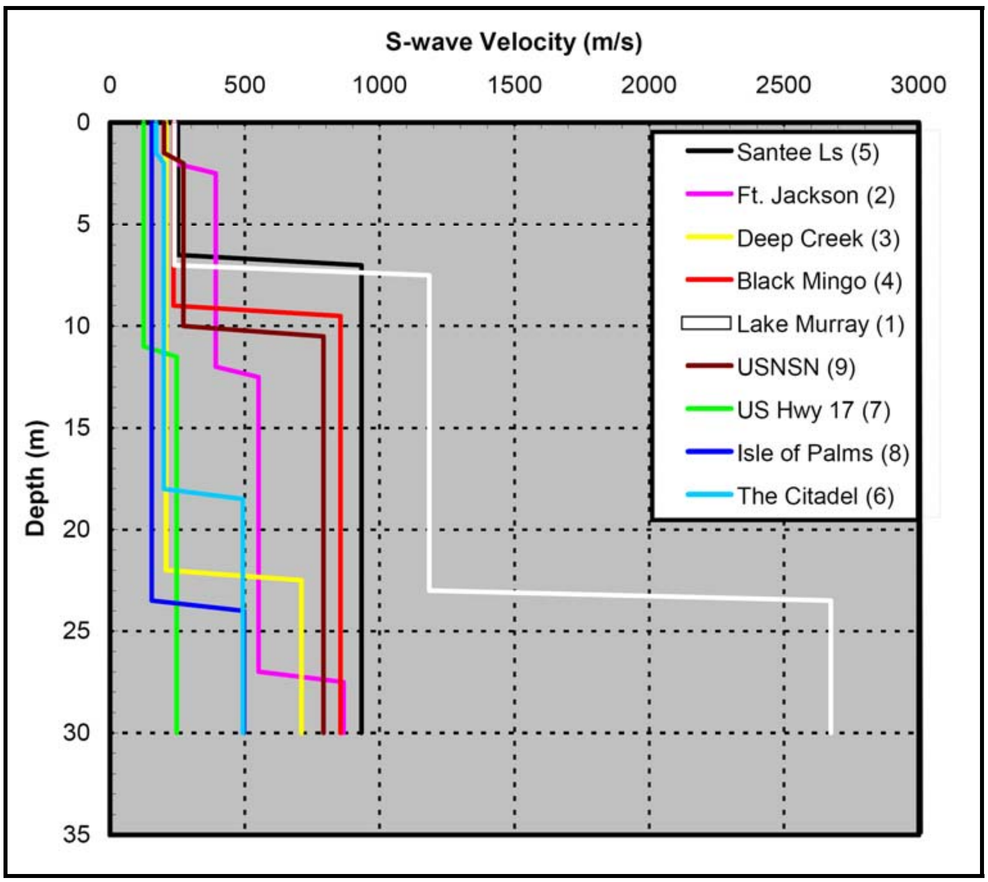


Figure 12-7, USGS Shear Wave V_S Profile (Odum et al., 2003)

The shear wave (V_S) and compression wave (V_P) profiles developed for the nine sites are shown in Figures 12-8 and 12-9. The columns show successively higher velocity layers V1, V2, and V3, indicated by yellow, blue, and light brown, respectively. For a detailed interpretation of the results shown in these profiles refer to Odum et al. (2003).

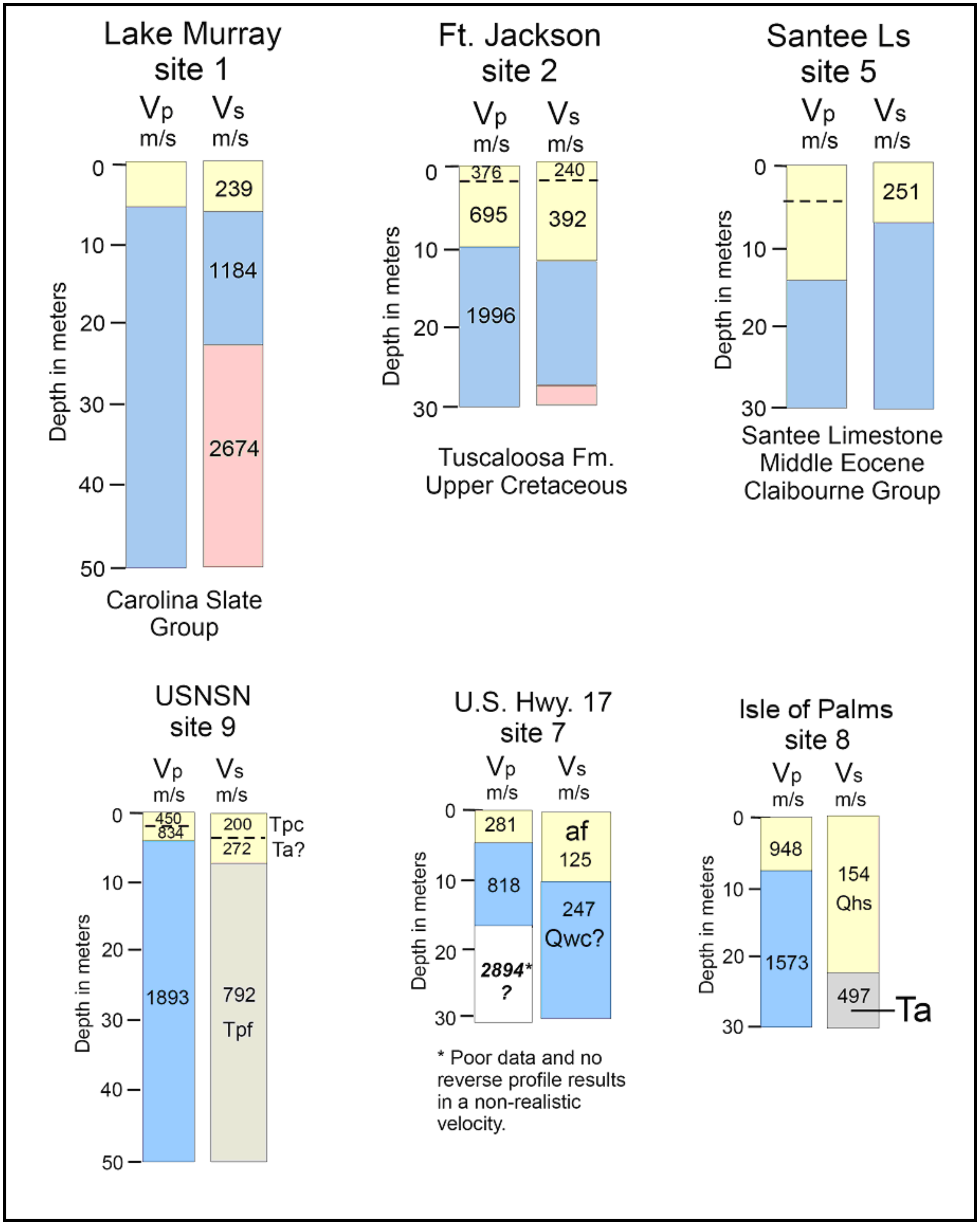


Figure 12-8, USGS Sites 1, 2, 5, 9, 7, and 8
(Odum et al., 2003)

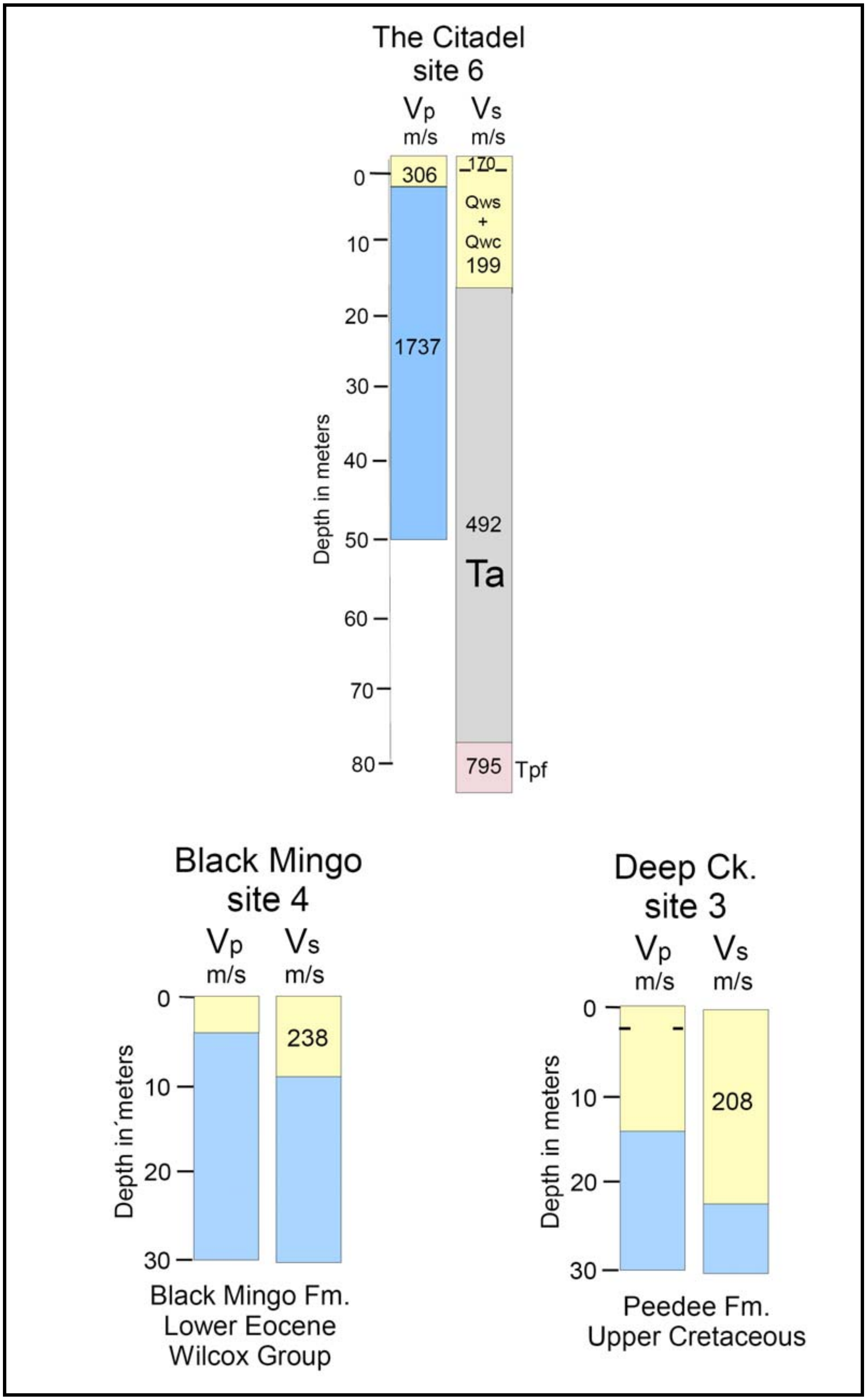


Figure 12-9, USGS Sites 6, 4, 3
(Odum et al., 2003)

12.3.3.2 SCEMD Seismic Risk and Vulnerability Study

A study was prepared by URS Corporation (2001) for the South Carolina Emergency Management Division (SCEMD). This study evaluated the potential losses resulting from four scenario earthquakes that may occur in South Carolina sometime in the future. South Carolina was divided into four site response categories based on physiographic provinces, surficial geology, and trends in subsurface data. The four site categories that were selected for this study are: Piedmont, Savannah River, Charleston, and Myrtle Beach. The extent of these site response categories are shown on a South Carolina map in Figure 12-10. The shear wave profiles for the Piedmont, Savannah River, Charleston, and Myrtle Beach are shown in Figures 12-11, 12-12, 12-13, and 12-14, respectively. For a detailed explanation of the base shear wave profiles used in this study refer to SCEMD report prepared by URS Corporation (2001).

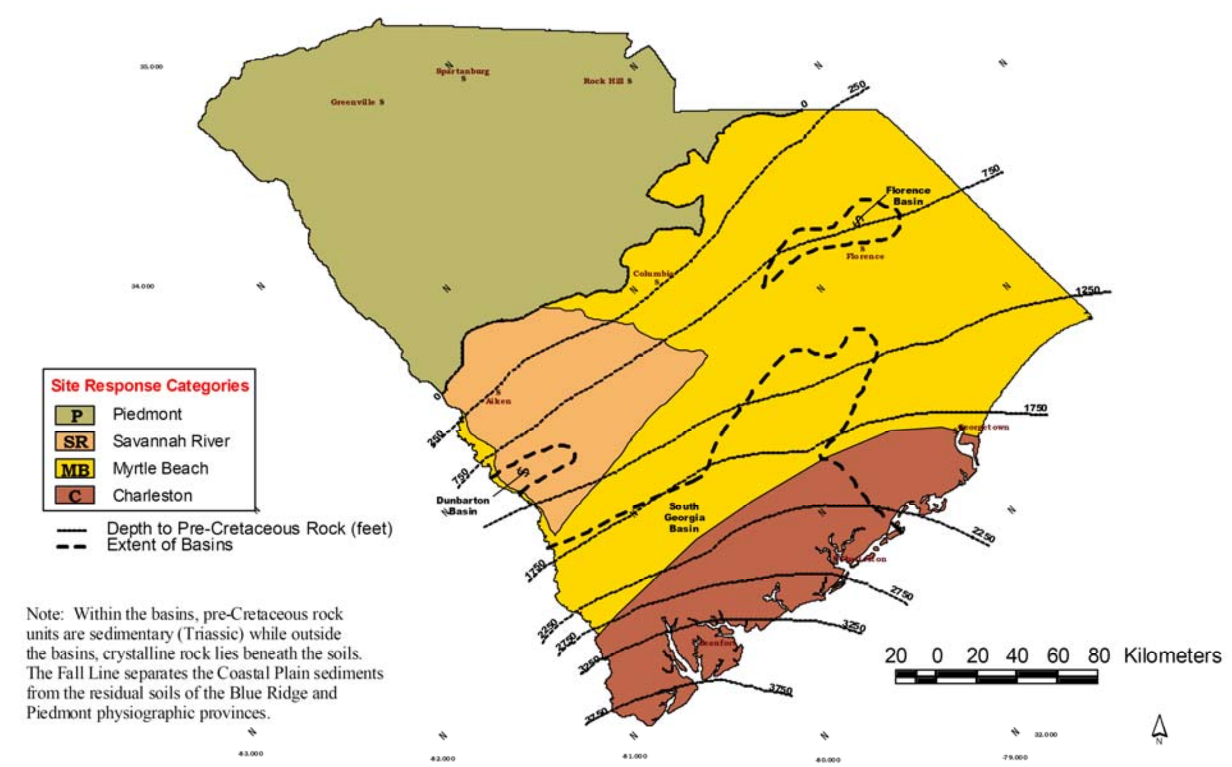


Figure 3-5. Site Response categories and depth to pre-Cretaceous rock.

Figure 12-10, Site Response Categories and Depth To Pre-Cretaceous Rock (URS Corporation, 2001)

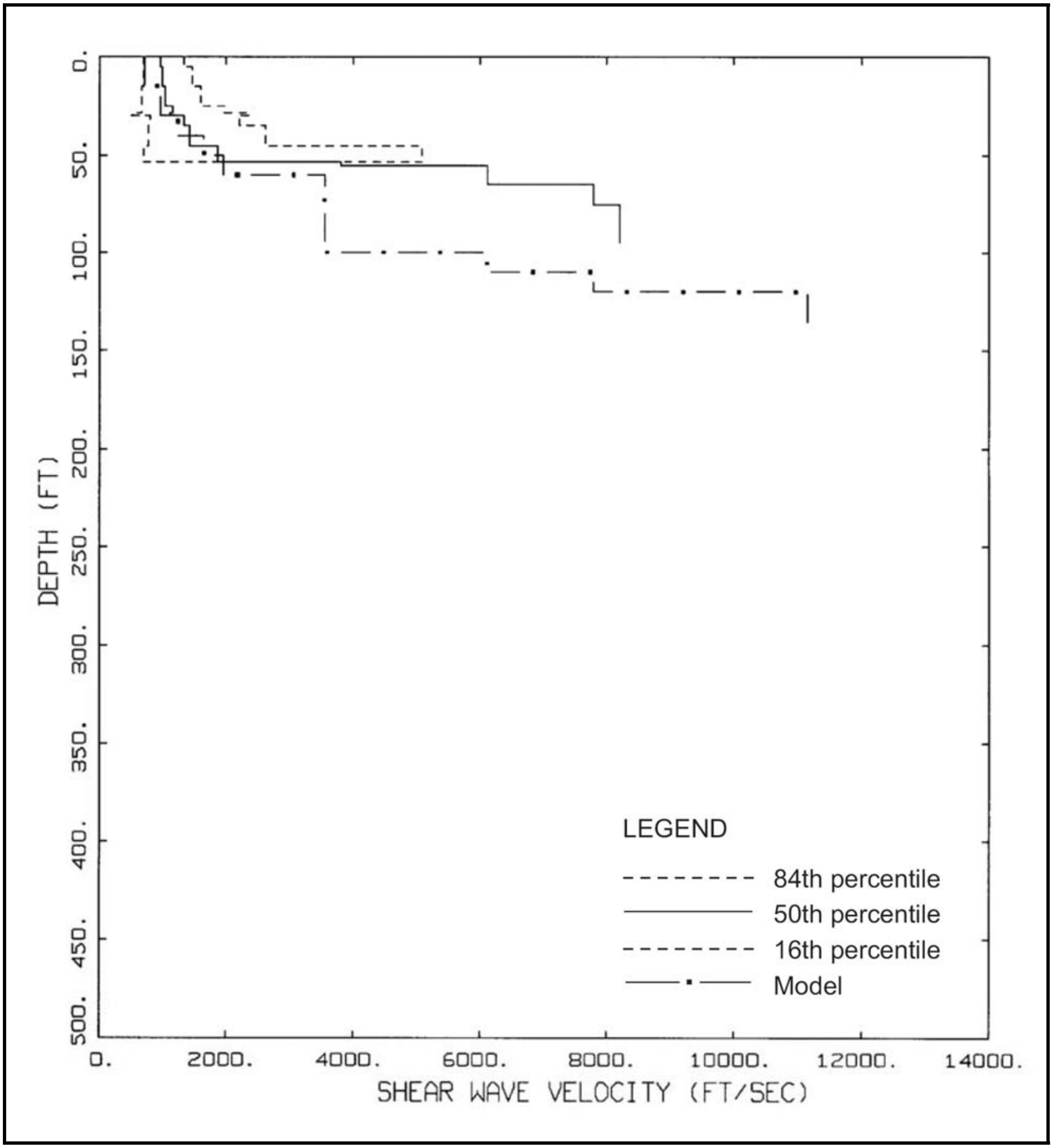


Figure 12-11, Piedmont/Blue Ridge Site Response Category Base Vs Profile (URS Corporation, 2001)

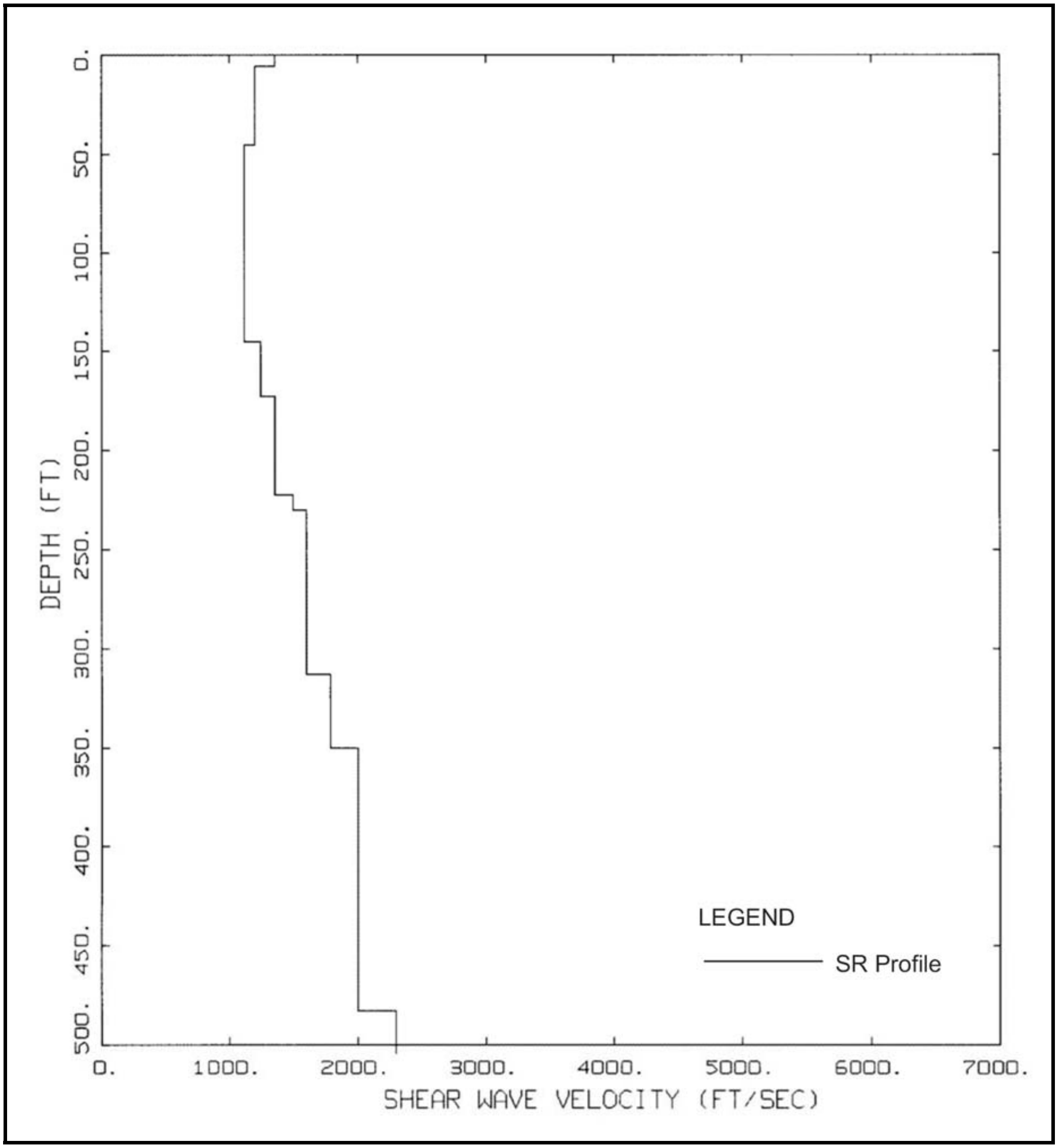


Figure 12-12, Savannah River Site Response Category Base Vs Profile (URS Corporation, 2001)

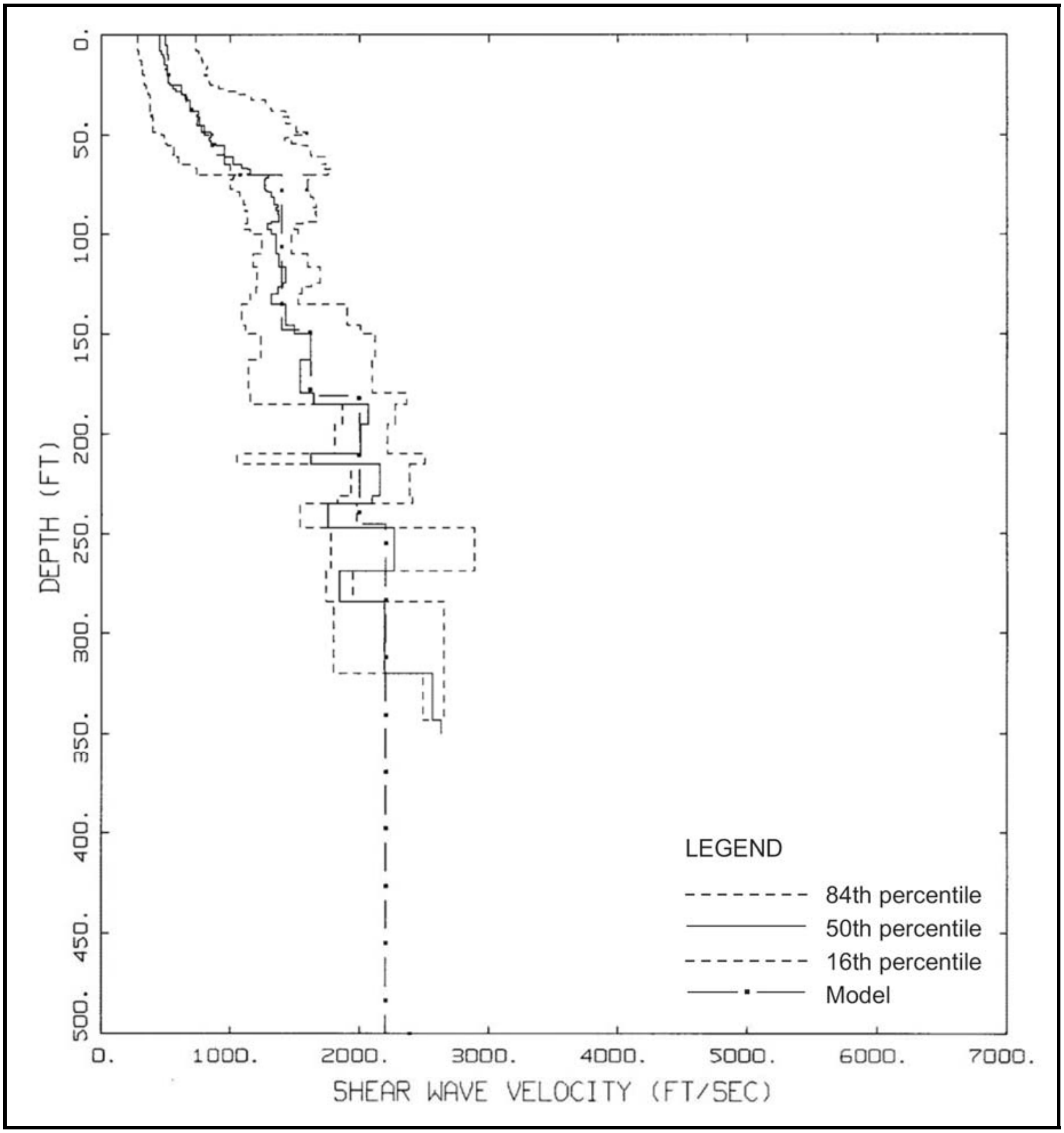


Figure 12-13, Charleston Site Response Category Base Vs Profile (URS Corporation, 2001)

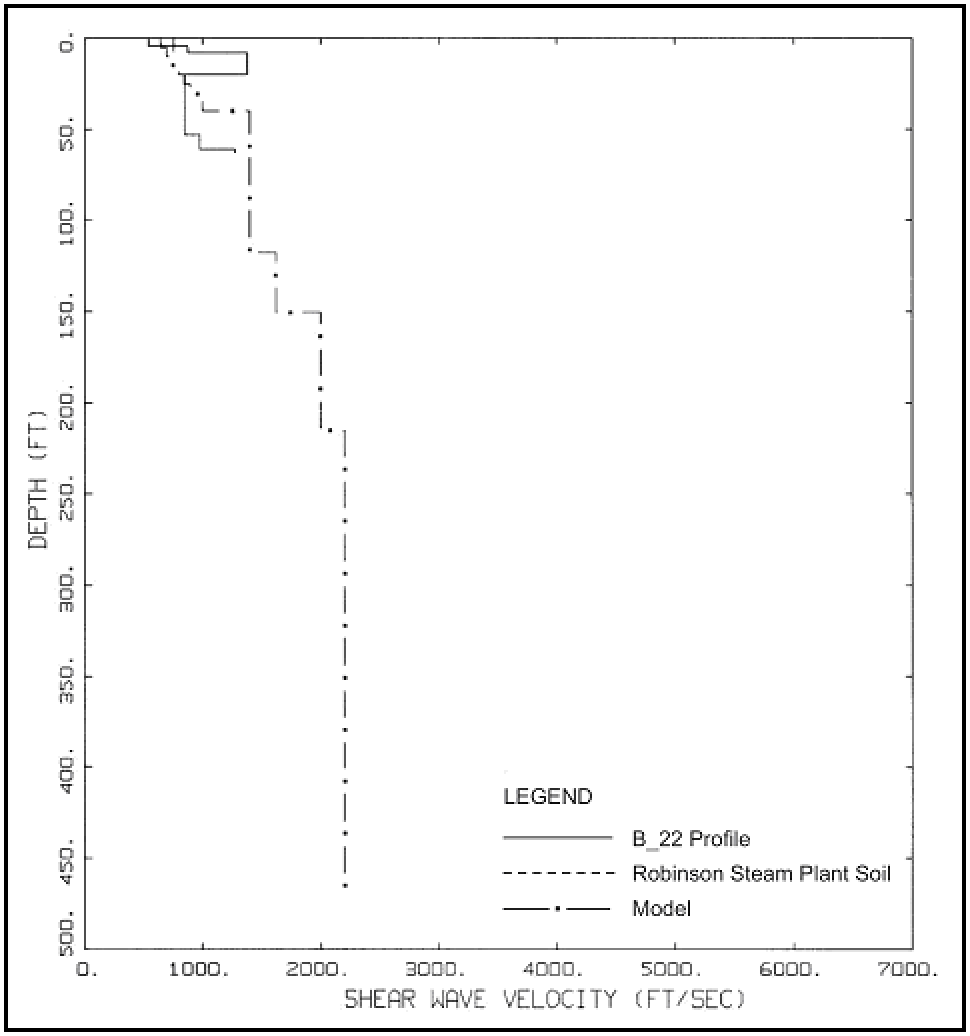


Figure 12-14, Myrtle Beach Site Response Category Base Vs Profile (URS Corporation, 2001)

12.3.3.3 Published / SCDOT Shear Wave Velocity Profiles

A review of published shear wave velocity profiles has been compiled to provide additional reference data for use in characterizing sites in South Carolina. The shear wave profiles are provided as references. For a detailed description of the geologic formation and geotechnical investigation, refer to the source documents. The list of the shear wave profiles compiled is provided below:

1. Seismic CPT and Geophysical shear wave profiles taken in Piedmont soils from the National Geotechnical Experimentation Sites (NGES) located at Opelika, Alabama. The Seismic CPT is shown in Figure 12-15 and the geophysical testing is shown in Figure 12-16. This site is generally accepted to be representative of Piedmont surface soils.

2. Seismic CPT shear wave profile taken at the Savannah River site in South Carolina is shown in Figure 12-17. This shear wave profile is generally representative of the soils at the U.S. Department of Energy Savannah River Site.

3. Seismic CPT shear wave profile taken at the Ravenel Bridge (Cooper River Bridge), located in Charleston, South Carolina, is shown in Figure 12-18.

4. Seismic CPT shear wave profiles were taken at Wetland Bridges 1 and 3 on US 17 between US Highway 21 intersection in Gardens Corner and the Combahee River. Two shear wave profiles were developed for Bridges 1 & 2 and Bridges 3 & 4 as shown in Figure 12-19. The SCPT B-14 taken at Bridge 1 is shown in Figure 12-20 and B-5A taken at Bridge 3 is shown in Figure 12-21.

5. Seismic CPT shear wave profiles were taken for a new bridge on US 378 over Great Pee Dee River, approximately 18 miles east of Lake City, South Carolina. Representative shear wave profiles from two SCPT SC3 and SC4 are shown in Figure 12-22 and 12-24, respectively. The corresponding SCPT logs for SC3 and SC4 are shown in Figures 12-23 and 12-25, respectively.

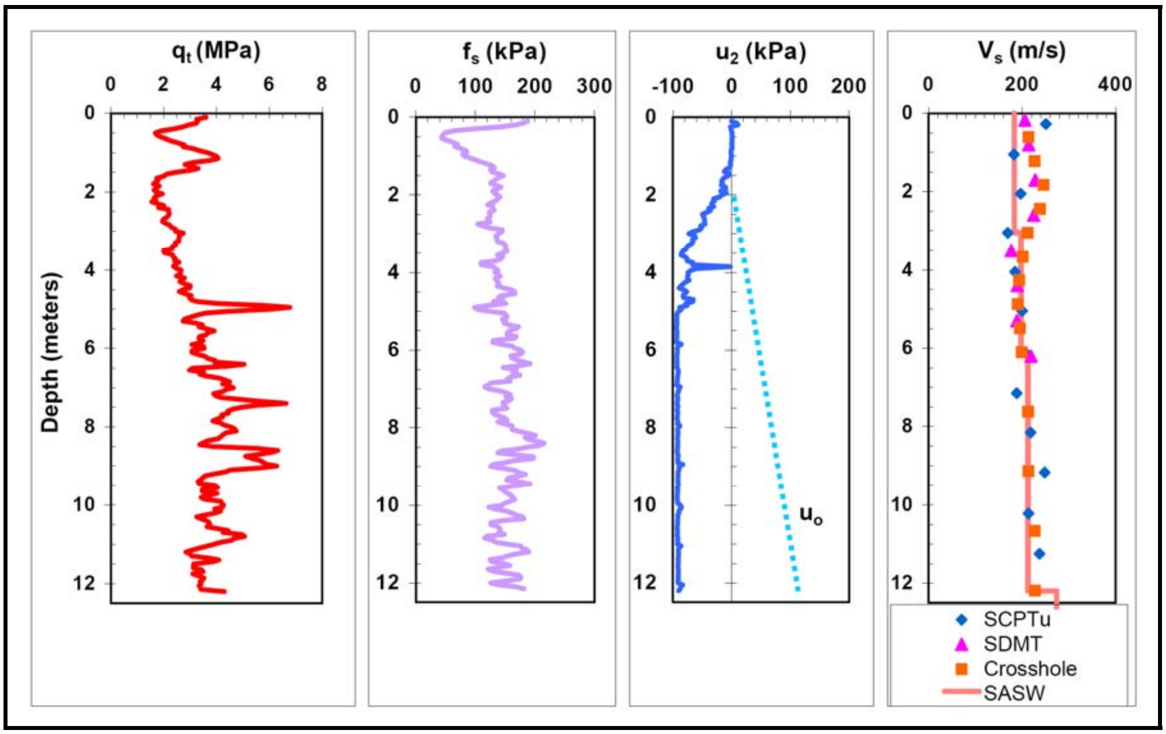


Figure 12-15, SCPT Piedmont Profile - NGES Opelika, Alabama (Mayne et al., 2000)

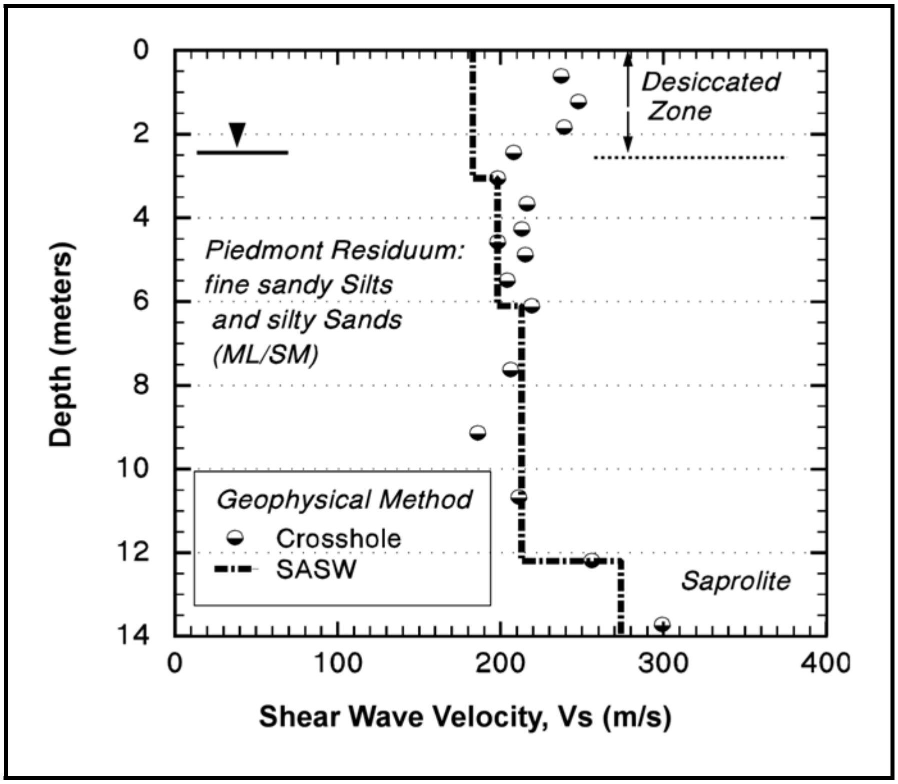


Figure 12-16, Geophysical V_S Piedmont Profile - NGES Opelika, Alabama (Mayne et al., 2000)

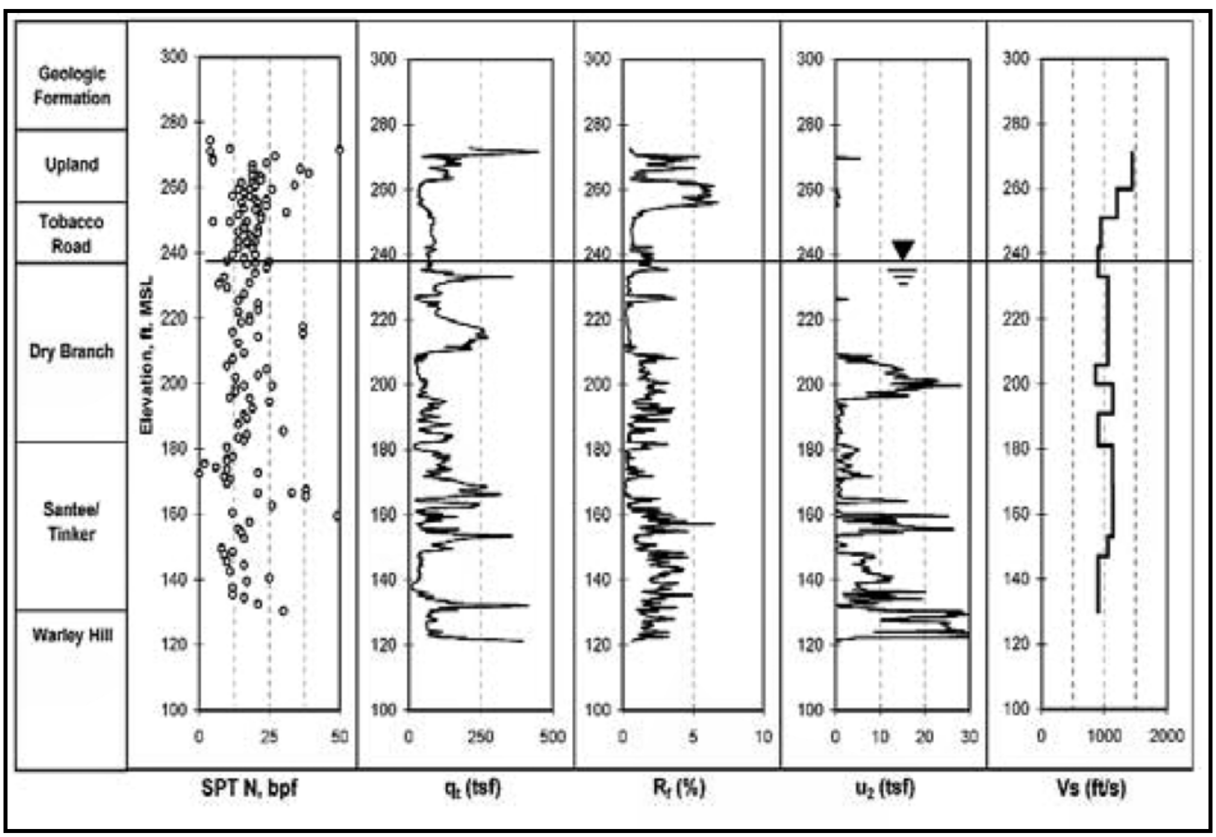


Figure 12-17, SCPT Profile Savannah River, South Carolina (Lewis et al., 2004)

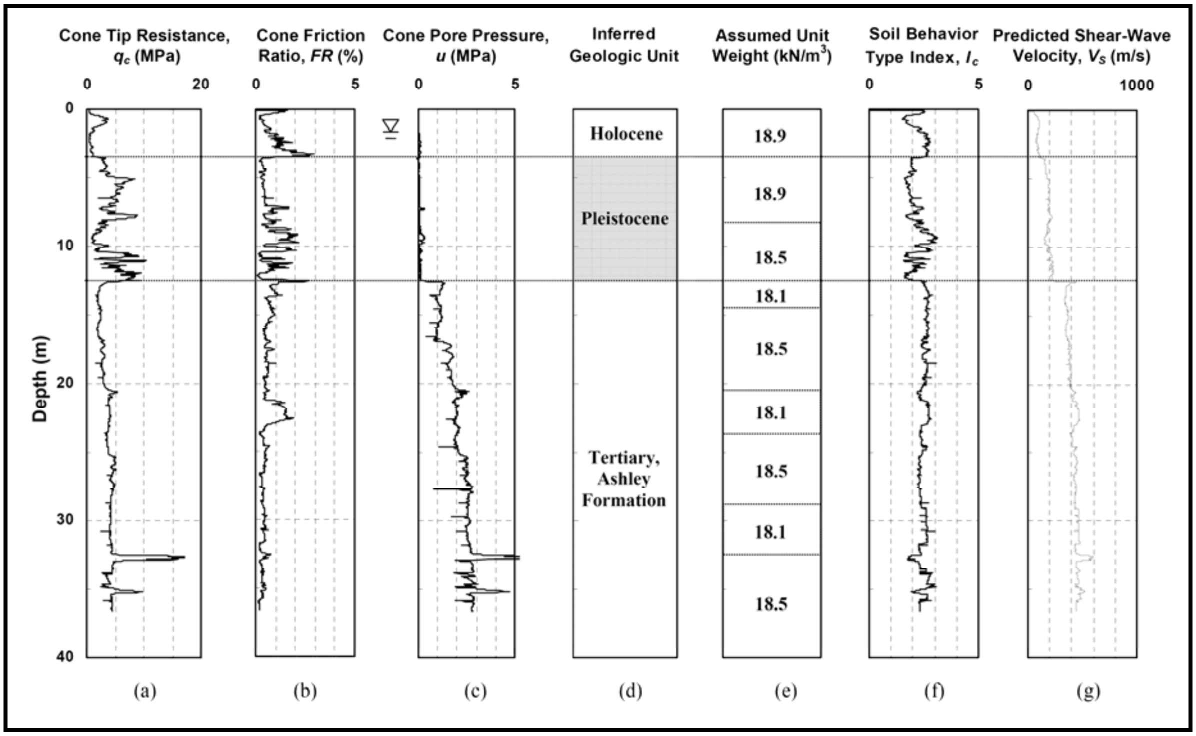


Figure 12-18, SCPT Profile (DS-1) Cooper River Bridge, Charleston, SC (S&ME, 2000)

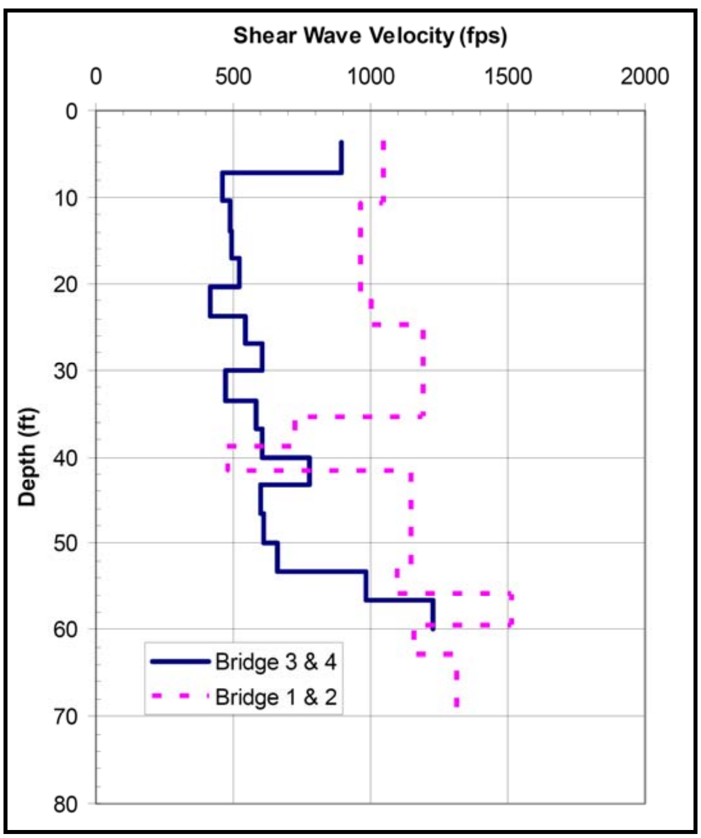


Figure 12-19, Shear Wave Profile US 17, Beaufort County, South Carolina (S&ME, 2007)

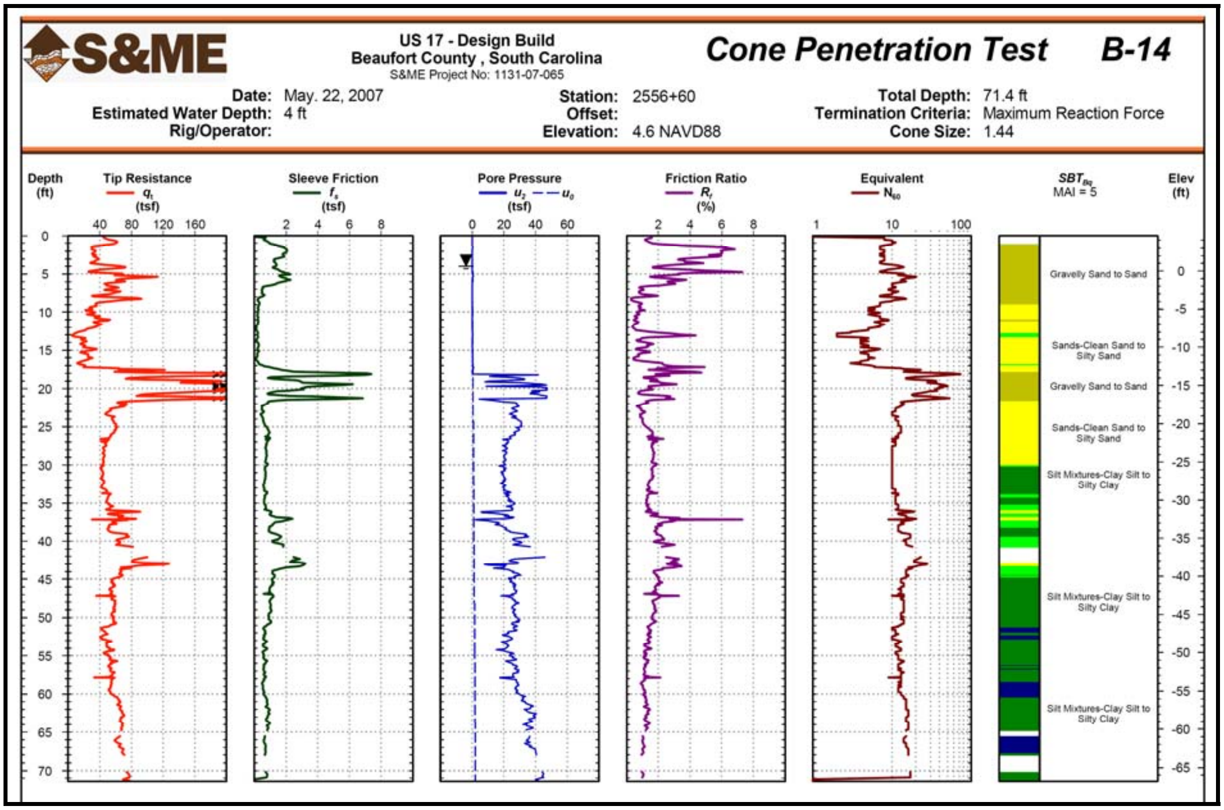


Figure 12-20, SCPT (B-14) US 17 Bridge 1, Beaufort County, South Carolina (S&ME, 2007)

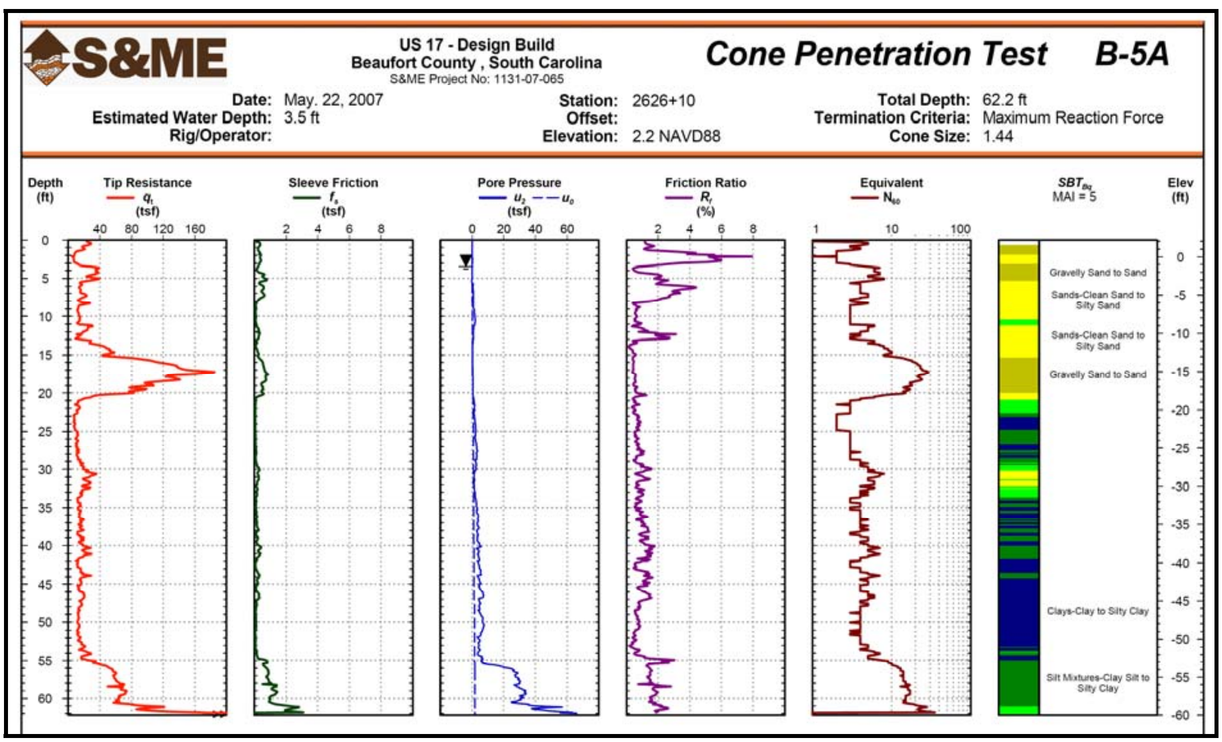


Figure 12-21, SCPT (B-5A) US 17 Bridge 3, Beaufort County, South Carolina (S&ME, 2007)

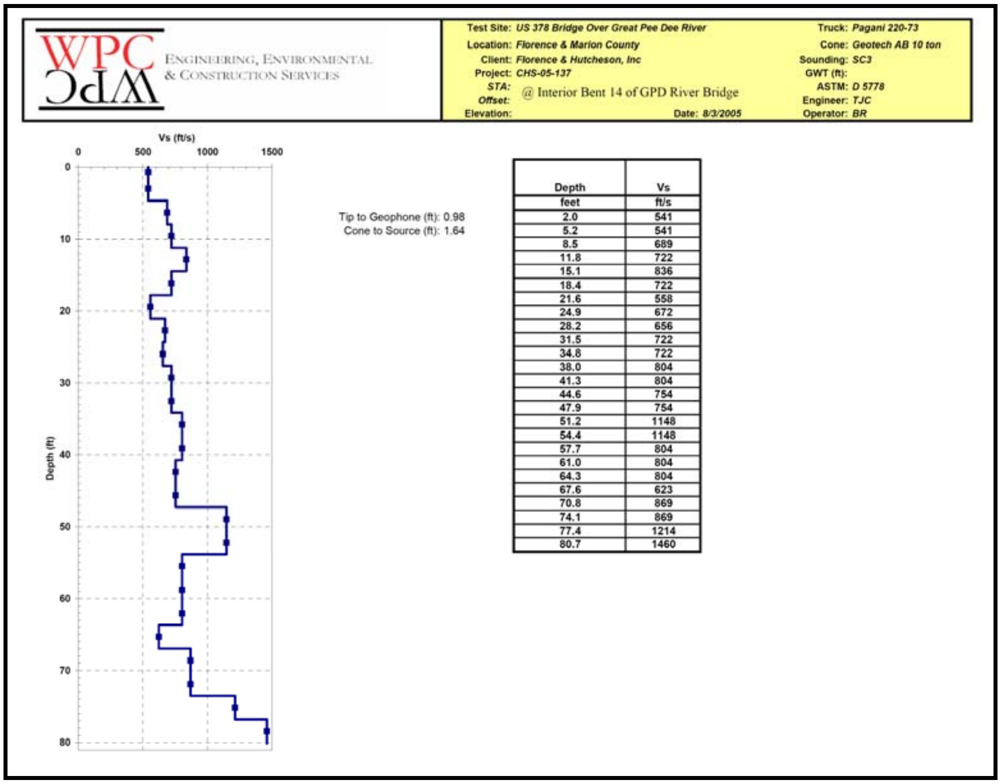


Figure 12-22, Shear Wave Profile (SC3) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)

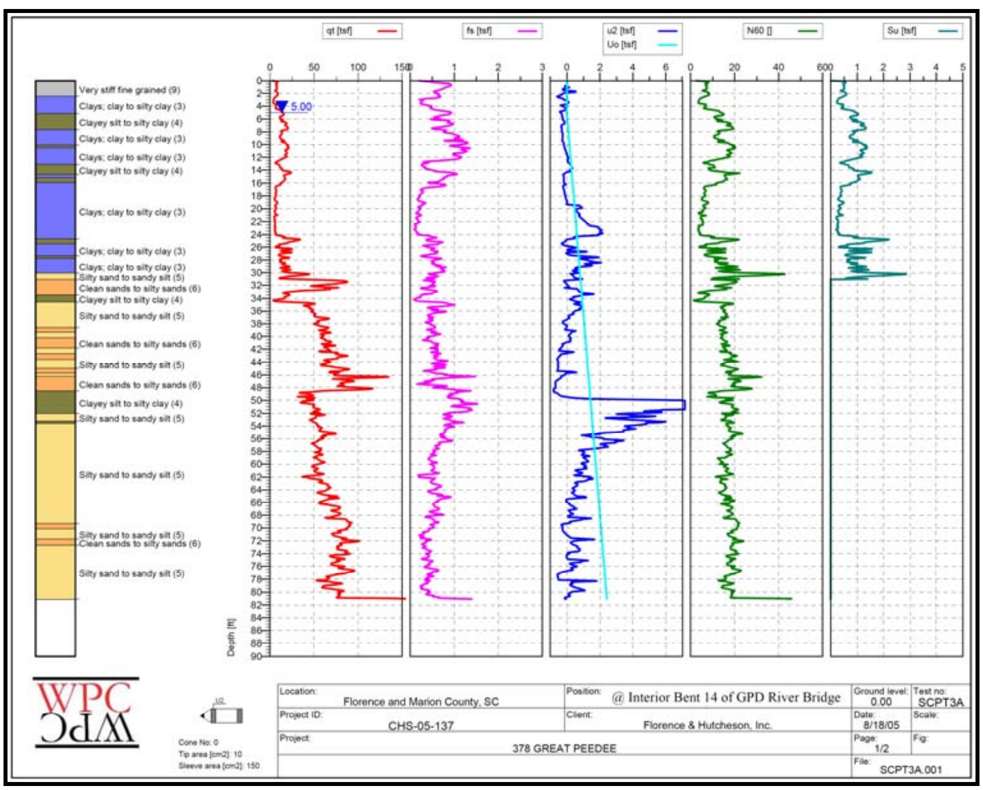
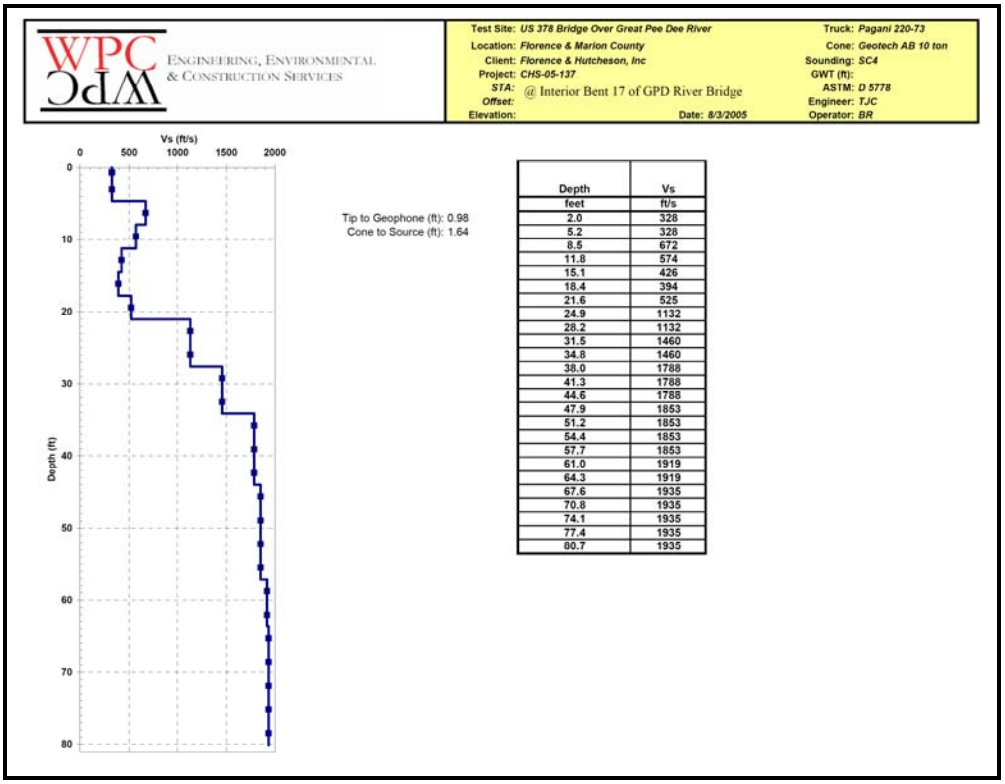


Figure 12-23, SCPT (SC3) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)



Depth	Vs
feet	ft/s
2.0	328
5.2	328
8.5	672
11.8	574
15.1	426
18.4	394
21.6	525
24.9	1132
28.2	1132
31.5	1460
34.8	1460
38.0	1788
41.3	1788
44.6	1788
47.9	1853
51.2	1853
54.4	1853
57.7	1853
61.0	1919
64.3	1919
67.6	1935
70.8	1935
74.1	1935
77.4	1935
80.7	1935

Figure 12-24, Shear Wave Profile (SC4) - US 378, Lake City, South (Florence & Hutcheson, 2006)

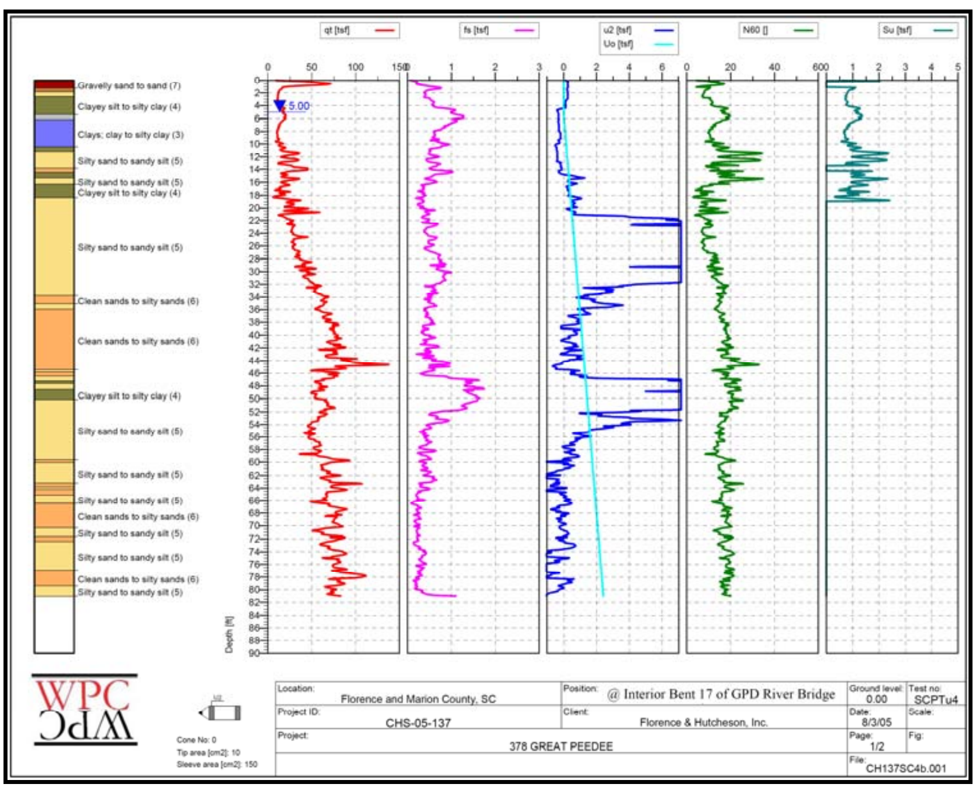


Figure 12-25, SCPT (SC4) - US 378, Lake City, South Carolina (Florence & Hutcheson, 2006)

12.3.4 Site Stiffness

Site stiffness is a measure of the overall soil stiffness (Section 12.3.2) of the soil layers to a specific depth of interest. Site stiffness in this Manual is computed as the weighted average of the shear wave velocities over a prescribed depth of the soil profile. The shear wave velocities, V_S, are not corrected for overburden. The weighted average can be computed by either using measured shear wave velocities obtained during the geotechnical site investigation or by using correlated shear wave velocities obtained from in-situ tests such as the Standard Penetration Test (SPT) and Cone Penetrometer Test (CPT) as indicated in previous sections.

Site stiffness in ft/sec (m/s) can be computed from measured shear wave velocities as indicated in the following equation.

$$\textit{Site Stiffness} = \frac{d_T}{t_d} \qquad \text{Equation 12-13}$$

Where,

d_T = total depth where shear wave velocities are being averaged in feet (m)

t_d = time that it takes for the shear wave to travel from the d_T to the ground surface (seconds)

Site stiffness in ft/sec (m/s) can also be computed by

$$\textit{Site Stiffness} = \frac{d_T}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}} \qquad \text{Equation 12-14}$$

Where,

d_T = total depth where shear wave velocities are being averaged in feet (m)

V_{si} = shear wave velocity of layer *i* in ft/sec (m/s)

d_i = thickness of any layer *i* between 0 and d_t

The distance below the ground surface, d_T, where the weighted shear wave velocities are computed is dependent on the type of geotechnical earthquake engineering analysis being performed. Consequently, site stiffnesses are designated and defined differently based on the depth of the zone of influence that shear wave velocity has on the computations that are being performed. The criteria for computing site stiffness for different types of geotechnical engineering correlations are provided in Table 12-12.

Table 12-12, Site Stiffness Definitions

Geotechnical Earthquake Engineering Correlation	Section Referenced	Site Stiffness Designation	d_T
Site Class Determination	12.4	$\bar{V}_s$	100 ft (30 m) below Z_{DTM} (1)
Nonlinear shear mass participation factor (r_d).	13.10.1	$V^*_{S,40'}$	40 feet (12 m)

(1) Z_{DTM} = Depth-to-motion. Additional guidance in determining d_T is provided in Sections 12.4.1 and 12.4.2.

12.3.5 Equivalent Uniform Soil Profile Period and Stiffness

The equivalent uniform soil profile period, T^*, and equivalent uniform soil profile stiffness, $V^*_{S,H}$, are used to compute the natural period of the site, T_N, as indicated in Section 12.6. The thickness of the profile (H) begins at the depth where the ground motion is of interest to the structure being designed (See Depth-to-Motion, Section 12.4.2) and extends to the depth where the motion is being generated, typically either the B-C boundary or a hard-rock basement outcrop (see Chapter 11). A comprehensive evaluation of how to determine the fundamental period of the soil profile has been made by Dobry et al. (1976). A simple and accurate method to determine the fundamental period of the soil profile is the Successive Two Layer Approach proposed by Madera (1970).

The Successive Two Layer Approach consists of solving for the fundamental period of two soil layers at a time, and then repeating the procedure successively (from the top to bottom of profile) until the entire soil profile is modeled as a single equivalent layer having a fundamental period, T^*. The Successive Two Layer Approach to compute the equivalent uniform soil profile period, T^*, and stiffness, $V^*_{S,H}$, is provided in Table 12-13.

Table 12-13, Successive Two Layer Approach

Step	Procedure Description
1	Begin with the layer at the top (n=1) of the profile under evaluation and continue working to the bottom of the profile. Compute the periods, T_A and T_B for top soil layers *A* (n) and bottom soil layer *B* (n + 1) using the following equations: $$T_A = \frac{4H_A}{V_{SA}}$$ Equation 12-15 $$T_B = \frac{4H_B}{V_{SB}}$$ Equation 12-16 Where, H_A = thickness of layer *A* in feet (m) H_B = thickness of layer *B* in feet (m) V_{SA} = shear wave velocity of layer *A* in ft/sec (m/s) V_{SB} = shear wave velocity of layer *B* in ft/sec (m/s) n = soil layer number (where top layer n = 1)
2	Compute the ratio of H_A/H_B and T_B/T_A.
3	Obtain the ratio of the uniform period, *T*, to T_A (T/T_A) for the combined two-layer system using Figure 12-26. Where T is the fundamental period for the two-layer
4	Compute the fundamental period, T, of the two layer system (A + B) using the following equation. $$T = \left(\frac{T}{T_A}\right) T_A$$ Equation 12-17 Where, $\left(\frac{T}{T_A}\right)$ = Ratio obtained from Figure 12-26 T_A = fundamental period of layer *A* in seconds (Equation 12-15)
5	Repeat items 1 through 4, where the combined two-layer system from step 4 becomes layer A with a fundamental period, $T_A = T$. Continue successively until the entire soil profile has been evaluated and there is a single fundamental period, *T*, for the entire soil profile. At this time, the single fundamental period, *T*, for the entire soil profile becomes equal to the equivalent uniform soil profile period, T^*.
6	Compute the equivalent uniform soil profile stiffness, $V^*_{S,H}$, from the following equation. $$V^*_{S,H} = \frac{4H}{T^*}$$ Equation 12-18 Where, *H* = thickness of the entire soil profile layer in feet (m) T^* = equivalent uniform soil profile period in seconds (Step 5)

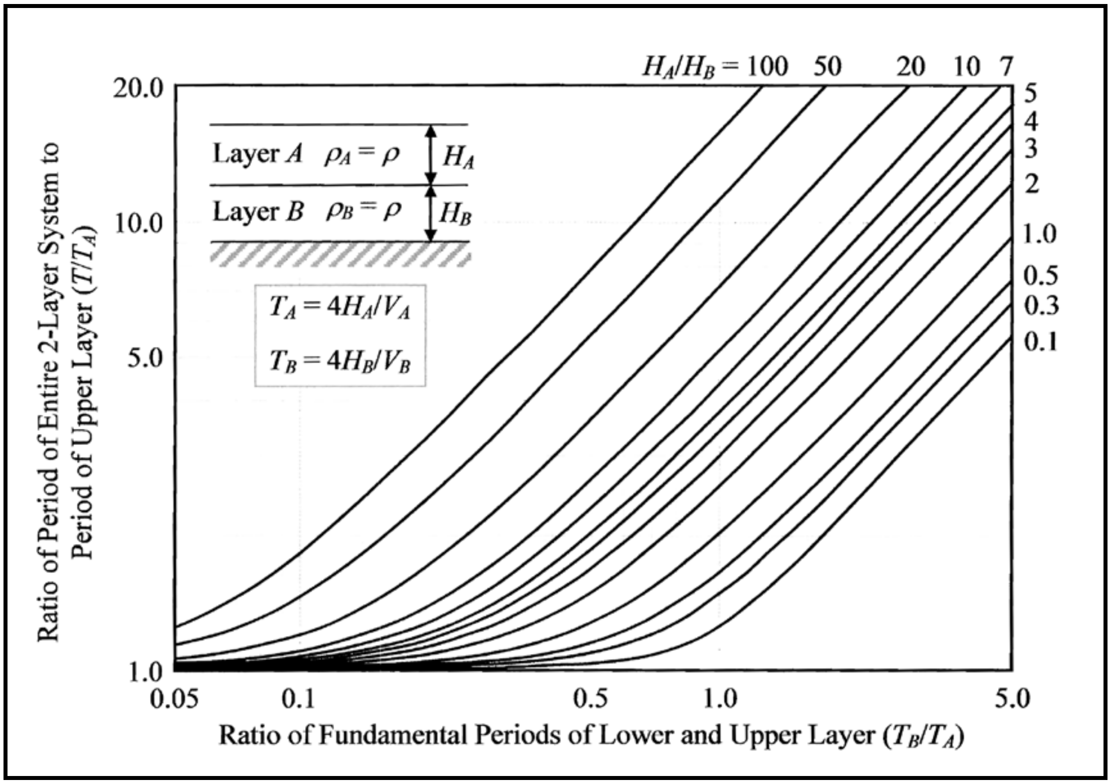


Figure 12-26, Fundamental Period of Two-Layer System (Oweis et al., 1975, Adapted by Green (2001))

12.3.6 Shear Modulus Reduction Curves

Shear modulus reduction curves are typically presented as normalized shear modulus, G/G_{max} vs. shear strain, γ. These curves are used for performing site-specific response analyses. These shear modulus reduction curves are primarily influenced by the strain amplitude, confining pressure, soil type, and plasticity. The shear modulus reduction curve is typically obtained by using a hyperbolic model. A modified hyperbolic model by Stokoe et al. (1999) has been used by Andrus et al. (2003) to develop shear modulus reduction curves for South Carolina soils. The hyperbolic model by Stokoe et al. (1999) is shown in the following equation.

$$G/G_{max} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}} \quad \text{Equation 12-19}$$

Where, α is the curvature coefficient, γ, is the shear strain, and γ_r is the reference shear strain. The curvature coefficient, α, and reference shear strain, γ_r, have been estimated by Andrus et al. (2003) to provide the most accurate values for South Carolina Soils. Because it was found that the reference shear strain, γ_r, varied based on effective confining pressure, reference shear strain, γ_r, values are computed using reference shear strain at 1 tsf (100 kPa, 1 atm), γ_{r1}, as shown in the following equation.

$$\gamma_r = \gamma_{r1}\left(\sigma'_m/P_a\right)^k \quad \text{Equation 12-20}$$

The mean confining pressure, σ'_m, at depth (Z) is computed as shown in Equation 12-21 in units of kPa, where P_a is the reference pressure of 100 kPa, and k is an exponent that varies based on the geologic formation and Plasticity Index, *PI*. Laboratory studies by Stokoe et al. (1995) indicate that the mean confining pressure, σ'_m, values of each layer within a geologic unit should be within ±50 percent of the range of σ'_m for the major geologic unit.

$$\sigma'_m = \sigma'_v \left[\frac{1+2K'_o}{3}\right] \qquad \text{Equation 12-21}$$

Where,

σ'_v = vertical effective pressure (kPa)

K'_o = coefficient of effective earth pressure at rest. The K'_o is defined as the ratio of horizontal effective pressure, σ'_h, to vertical effective pressure, σ'_v. The coefficient of effective earth pressure at-rest, K'_o, can be approximated by the coefficient of at-rest pressure, K_o, equations shown in Table 12-14.

Table 12-14, Estimated Coefficient of At-Rest Pressure, K_o

Soil Type	Equation [1]	Equation No.
Normally Consolidated Granular Soils (Jaky, 1944)	$K_0 \approx 1-\sin\phi'$	Equation 12-22
Normally Consolidated Clay Soils (Brooker and Ireland, 1965)	$K_0 \approx 0.95-\sin\phi'$	Equation 12-23
Normally Consolidated Clay Soils (0 < PI ≤ 40) (Brooker and Ireland, 1965)	$K_o \approx 0.40+0.007(PI)$	Equation 12-24
Normally Consolidated Clay Soils (40 < PI < 80) (Brooker and Ireland, 1965)	$K_o \approx 0.6+0.001(PI)$	Equation 12-25
Overconsolidated Clays (Alpan, 1967; Schmertmann, 1975)	$K_o \approx K_{o(N.C.)}\sqrt{OCR}$	Equation 12-26
Overconsolidated Soils (Mayne and Kulhawy, 1982)	$K_o \approx K_{o(N.C.)}OCR^{\sin\phi'}$	Equation 12-27

[1] ϕ'=Drained Friction Angle; *PI*=Plasticity Index; *N.C.*=Normally Consolidated; *OCR* = Overconsolidated Ratio

Values for the reference strain at 1 tsf (100 kPa, 1 atm), γ_{r1}, curvature coefficient, α, and k exponent are provided for South Carolina soils based on Andrus et al. (2003) in Table 12-15.

Table 12-15, Recommended Values γ_{r1}, α, and k for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits [1]	Variable	Soil Plasticity Index, *PI* (%)					
		0	15	30	50	100	150
Holocene	γ_{r1} (%)	0.073	0.114	0.156	0.211	0.350	0.488
	α	0.95	0.96	0.97	0.98	1.01	1.04 [2]
	k	0.385	0.202	0.106	0.045	0.005	0.001 [2]
Pleistocene (Wando)	γ_{r1} (%)	0.018	0.032	0.047	0.067	0.117	0.166
	α	1.00	1.02	1.04	1.06	1.13	1.19
	k	0.454	0.402	0.355	0.301	0.199	0.132
Tertiary Ashley Formation (Cooper Marl)	γ_{r1} (%)	---	---	0.030 [2]	0.049	0.096 [2]	---
	α	---	---	1.10 [2]	1.15	1.28	---
	k	---	---	0.497 [2]	0.455	0.362 [2]	---
Tertiary (Stiff Upland Soils)	γ_{r1} (%)	---	---	0.023	0.041 [2]	---	---
	α	---	---	1.00	1.00 [2]	---	---
	k	---	---	0.102	0.045 [2]	---	---
Tertiary (All soils at SRS except Stiff Upland Soils)	γ_{r1} (%)	0.038	0.058	0.079	0.106	0.174 [2]	---
	α	1.00	1.00	1.00	1.00	1.00 [2]	---
	k	0.277	0.240	0.208	0.172	0.106 [2]	---
Tertiary (Tobacco Road, Snapp)	γ_{r1} (%)	0.029	0.056	0.082	0.117	0.205 [1]	---
	α	1.00	1.00	1.00	1.00	1.00 [1]	---
	k	0.220	0.185	0.156	0.124	0.070 [1]	---
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	γ_{r1} (%)	0.047	0.059	0.071	0.086	0.125 [1]	---
	α	1.00	1.00	1.00	1.00	1.00 [1]	---
	k	0.313	0.299	0.285	0.268	0.229 [1]	---
Residual Soil and Saprolite	γ_{r1} (%)	0.040	0.066	0.093 [1]	0.129 [1]	---	---
	α	0.72	0.80	0.89	1.01 [1]	---	---
	k	0.202	0.141	0.099	0.061 [2]	---	---

[1] SRS = Savannah River Site
[2] Tentative Values – Andrus et al. (2003)

The procedure for computing the G/G_{max} correlation using Equation 12-19 is provided in Table 12-16.

Table 12-16, Procedure for Computing G/G_{max}

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average *PI*, and soil density. Subdivide major geologic units to reflect significant changes in *PI* and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 12-21
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select the appropriate values for each layer of reference strain, γ_{r1}, at 1 tsf (1 atm), curvature coefficient, α, and *k* exponent from Table 12-15. These values may be selected by rounding to the nearest *PI* value in the table or by interpolating between listed *PI* values in the table.
6	Compute the reference strain, γ_r, based on Equation 12-20 for each geologic unit (or "subdivided" geologic unit) that has a corresponding average σ'_m.
7	Compute the design shear modulus reduction curves (G/G_{max}) for each layer by substituting reference strain, γ_r, and curvature coefficient, α, for each layer using Equation 12-19. Tabulate values of normalized shear modulus, G/G_{Max} with corresponding shear strain, γ, for use in a site-specific response analysis.

12.3.7 Equivalent Viscous Damping Ratio Curves

Equivalent Viscous Damping Ratio curves are presented in the form of a Soil Damping Ratio, *D* vs. Shear Strain, γ. The Soil Damping Ratio represents the energy dissipated by the soil and is related to the stress-strain hysteresis loops generated during cyclic loading. Energy dissipation or damping is due to friction between soil particles, strain rate effects, and nonlinear behavior of soils. The damping ratio is never zero, even when soils are straining within the linear elastic range of the cyclic loading. The damping ratio, *D*, is constant during the linear elastic range of the cyclic loading and is referred to as the small-strain material damping, D_{min}. The small-strain material damping, D_{min}, can be computed using Stokoe et al., (1995) Equation 12-28.

$$D_{\min} = D_{\min 1}\left(\sigma'_m / P_a\right)^{-0.5k} \qquad \text{Equation 12-28}$$

Where D_{min1} is the small-strain damping at a σ'_m of 1 tsf (1 atm). The mean confining pressure, σ'_m, is computed using Equation 12-21. The *k* exponent is provided for South Carolina soils based on Andrus et al. (2003) in Table 12-15. A relationship for D_{min1} based on soil plasticity index, *PI*, and fitting parameters "*a*" and "*b*" for specific geologic units has been developed by Darendeli (2001) as indicated in Figure 12-27. Values for D_{min1}, small-strain damping @ σ'_m = 1 atm are provided for South Carolina soils based on Andrus et al. (2003) in Table 12-17. The mean confining pressure, σ'_m, at depth (*Z*) is computed as shown in Equation 12-21 in units of kPa.

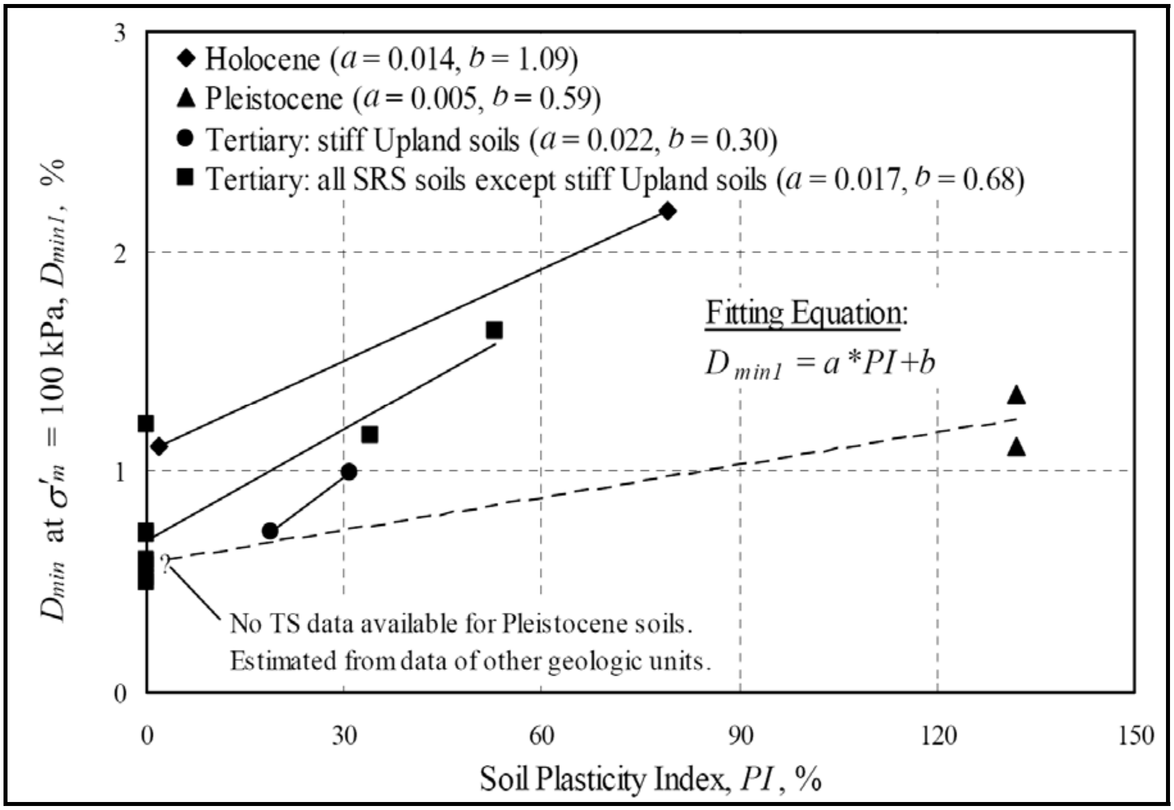


Figure 12-27, D_{min1}, Small-Strain Damping @ σ'_m = 1 atm (Andrus et al., 2003)

Table 12-17, Recommended Value D_{min1} (%) for SC Soils (Andrus et al., 2003)

Geologic Age and Location of Deposits	Soil Plasticity Index, *PI* (%)					
	0	**15**	**30**	**50**	**100**	**150**
Holocene	1.09	1.29	1.50	1.78	2.48	3.18 (1)
Pleistocene (Wando)	0.59	0.66	0.73	0.83	1.08	1.32
Tertiary Ashley Formation (Cooper Marl)	---	---	1.14 (1)	1.52 (1)	2.49 (1)	---
Tertiary (Stiff Upland Soils)	---	---	0.98	1.42 (1)	---	---
Tertiary (All soils at SRS except Stiff Upland Soils)	0.68	0.94	1.19	1.53	2.37 (1)	---
Tertiary (Tobacco Road, Snapp)	0.68	0.94	1.19	1.53	2.37 (1)	---
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	0.68	0.94	1.19	1.53	2.37 (1)	---
Residual Soil and Saprolite	0.56 (1)	0.85 (1)	1.14 (1)	1.52 (1)	---	---

(1) Tentative Values – Andrus et al. (2003)

Data compiled by the University of Texas at Austin (UTA) for (D – D_{min}) vs. (G/G_{max}) is plotted in Figure 12-28.

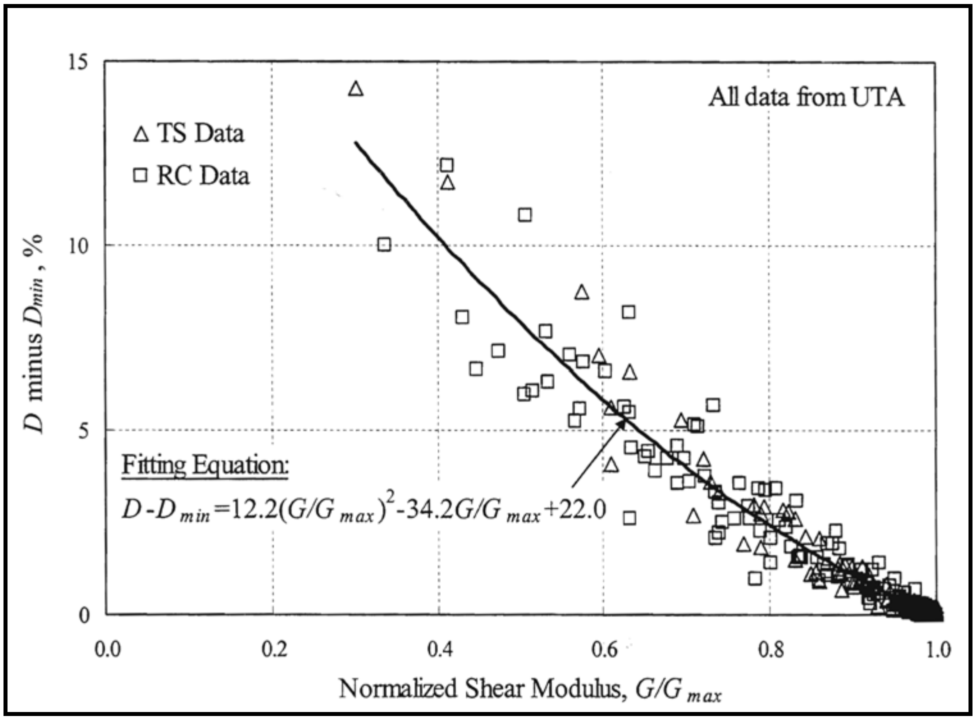


Figure 12-28, (D – D_{min}) vs. (G/G_{max}) Relationship (Andrus et al., 2003)

Equation 12-29 represents a best-fit equation (UTA Correlation) of the observed relationship of (D – D_{min}) vs. (G/G_{max}) indicated in Figure 12-28.

$$D - D_{min} = 12.2\left(G/G_{max}\right)^2 - 34.2\left(G/G_{max}\right) + 22.0 \qquad \text{Equation 12-29}$$

If we substitute Equation 12-19 into Equation 12-29 and Solve for damping ratio, D, the Equivalent Viscous Damping Ratio curves can be generated using Equation 12-30.

$$D = D_{min} + 12.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right)^2 - 34.2\left(\frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}}\right) + 22.0 \qquad \text{Equation 12-30}$$

Where values of reference strain, γ_r, are computed using Equation 12-20.

The procedures for using Equation 12-30 are provided in Table 12-18.

Table 12-18, Procedure for Computing Damping Ratio

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average *PI*, and soil density. Subdivide major geologic units to reflect significant changes in *PI* and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 12-21.
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select appropriate small-strain material Damping @ σ'_m = 1 atm, D_{min1}, from Table 12-17 for each layer within a geologic unit.
6	Compute the small-strain material Damping, D_{min}, for each layer within a geologic unit using Equation 12-28.
7	Select the appropriate values for each layer of reference strain, γ_{r1}, @ σ'_m = 1atm , curvature coefficient, α, and *k* exponent from Table 12-15. These values may be selected by rounding to the nearest *PI* value in the table or by interpolating between listed *PI* values in the table.
8	Compute the reference strain, γ_r, based on Equation 12-20 for each geologic unit that has a corresponding average σ'_m.
9	Compute the design equivalent viscous damping ratio curves (*D*) for each layer by substituting reference strain, γ_r, and curvature coefficient, α, and small-strain material Damping, D_{min}, for each layer using Equation 12-30. Tabulate values of Soil Damping Ratio, *D*, with corresponding shear strain, γ, for use in a site-specific site response analysis.

12.3.8 Alternate Dynamic Property Correlations

12.3.8.1 Soil Stiffness

The SPT and CPT shear wave, V_s, correlations provided in Sections 12.3.2.1 and 12.3.2.2 are based on studies performed by Andrus et al. (2003) for South Carolina soils. If the Andrus et al. (2003) shear wave correlations are not appropriate (i.e. embankment fill) for the soils encountered at a specific project site, the geotechnical engineer can use alternate correlations provided that documentation is provided explaining the use of an alternate correlation and that the correlation is nationally or regionally recognized. Acceptable correlations that can be used are listed in Table 12-19.

Table 12-19, Alternate Correlations of Soil Stiffness (G_{max})

Reference	Correlation Equation	Units	Comments
Seed, et al. (1984)	$G_{max} = 220(K_2)_{max}(\sigma'_m)^{0.5}$ $(K_2)_{max} \approx 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max} \approx$ 30 for loose sands and 75 for very dense sands; ≈80-180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560(N_{60})^{0.68}$	kPa	Limited to cohesionless soils
Hardin (1978)	$G_{max} = \frac{625}{(0.3 + 0.7e_o^2)}(P_a\sigma'_m)^{0.5}OCR^k$	kPa [1]	Limited to cohesive soils P_a = atmospheric pressure P_a and σ'_m in kPa
Jamiolkowski, et al. (1991)	$G_{max} = \frac{625}{e_o^{1.3}}(P_a\sigma'_m)^{0.5}OCR^k$	kPa [1]	Limited to cohesive soils P_a and σ'_m in kPa
Mayne and Rix (1993)	$G_{max} = 99.5(P_a)^{0.305}\frac{(q_c)^{0.695}}{(e_o)^{1.13}}$	kPa	Limited to cohesive soils P_a and q_c in kPa

[1] The parameter k is related to the plasticity index, PI, as follows:

PI	***k***	***PI***	***k***
0	0.00	60	0.41
20	0.18	80	0.48
40	0.30	>100	0.50

12.3.8.2 Shear Modulus Reduction Curves

The shear modulus reduction curves provided in Section 12.3.6 are based on studies performed by Andrus et al. (2003) for South Carolina soils. If the Andrus et al. (2003) shear modulus reduction curves are not appropriate (i.e. embankment fill) for the soils encountered at a specific project site, the geotechnical engineer may use alternate shear modulus reduction curve correlations provided that documentation is provided explaining the use of the alternate curve and that the alternate curve is nationally or regionally recognized. Acceptable correlations that may be used are listed below:

- Seed and Idriss (1970)
- Vucetic and Dobry (1991)
- Ishibashi and Zhang (1993)
- Idriss (1990)
- Seed et al. (1986)

12.3.8.3 Equivalent Viscous Damping Ratio Curves

The equivalent viscous damping ratio curves provided in Section 12.3.7 are based on studies performed by Andrus et al. (2003) for South Carolina soils. If the Andrus et al. (2003) equivalent viscous damping ratio curves are not appropriate (i.e. embankment fill) for the soils encountered at a project site the geotechnical engineer may use alternate equivalent viscous damping ratio curves provided that documentation is provided explaining the use of the alternate curve and that the alternate curve is nationally or regionally recognized. Acceptable correlations that may be used are listed below:

- Seed et al. (1986)
- Idriss (1990)
- Vucetic and Dobry (1991)

12.4 PROJECT SITE CLASSIFICATION

12.4.1 Site Class Determination

The first step in earthquake engineering is to categorize the project site based on the Site Class. The Site Class of a project site is determined by assigning a Site Class of A, B, C, D, E, or F based on the site stiffness, $\bar{V}_s$, criteria provided in Table 12-22. The site stiffness is a weighted average of the shear wave velocities at the project site. The geotechnical engineer-of-record determines the Site Class based on a careful evaluation of the subsurface investigation and field and laboratory testing results. The project Site Class is determined during the preliminary exploration through the collection of shear wave velocities (Chapters 4 and 5). If the Site Class is required and a preliminary subsurface investigation has not been performed, the geotechnical engineer may use geotechnical information available at the site, past subsurface investigations in the area, and consult geologic maps of the region. After the site-specific geotechnical subsurface investigation has been completed, the preliminary Site Class provided will be re-evaluated and a final Site Class will be provided if necessary.

The site stiffness, $\bar{V}_s$, should be computed in accordance with Section 12.3.4. The total depth (d_T) where shear wave velocities will be analyzed should begin at the anticipated depth-to-motion, Z_{DTM}, and extend to a depth of 100 feet (d_T = 100 ft.) or less if the soil column from the depth-to-motion, Z_{DTM}, to the location where the ground motion is placed using geologically realistic site conditions is located less than 100 feet. When evaluating Site Class C, D, E, or F, the soil column should consist of soils with shear wave velocities less than 2,500 ft/sec. The depth-to-motion is the location where the ground motion transmits the ground shaking energy to the structure being designed. Guidance in selecting the depth-to-motion, Z_{DTM}, is provided in Section 12.4.2.

When there is a high contrast in shear wave velocities in the soil column the computed site stiffness, $\bar{V}_s$, may not be representative of the site response. The geotechnical engineer will need to evaluate the computed site stiffness for high variation in shear wave velocity within the profile that could potentially overestimate the site stiffness and in turn underestimate amplification of the spectral accelerations. The following procedure to evaluate site stiffness, $\bar{V}_s$, variability is to be used cautiously as only a guide. The geotechnical engineer will be responsible for making all site stiffness, $\bar{V}_s$, recommendations, and these recommendations will be submitted to the PCS/GDS for approval. The proposed procedure to evaluate the site stiffness, $\bar{V}_s$, variability is based on the potential variability of shear wave testing having a Coefficient of Variability (COV) of 0.10 to 0.20. The proposed procedure to evaluate site stiffness variability is shown in Table 12-20.

Table 12-20, Site Stiffness Variability Proposed Procedure

Step	Description
1	Compute the Coefficient of Variability (COV) of the shear wave velocity values (COV_{VS}) within the soil profile column. If the COV_{VS} is greater than 0.10 proceed to Step 2. If the $COV_{VS} \leq 0.10$ then compute the site stiffness, $\overline{V}_s$, using the shear wave values (V_s) in accordance with Section 12.3.4 and then determine the Site Class.
2	If $0.10 < COV_{VS} \leq 0.20$ compute the adjusted site stiffness, $\overline{\boldsymbol{V}}_s^{'}$, using Equation 12-31 then proceed to Step 3. $$\overline{\boldsymbol{V}}_s^{'} = \overline{\boldsymbol{V}}_s(1 - 0.20) \qquad \text{Equation 12-31}$$ If $0.20 < COV_{VS} \leq 0.30$ compute the adjusted site stiffness, $\overline{\boldsymbol{V}}_s^{'}$, using Equation 12-32 then proceed to Step 3. $$\overline{\boldsymbol{V}}_s^{'} = \overline{\boldsymbol{V}}_s(1 - \boldsymbol{COV_{VS}}) \qquad \text{Equation 12-32}$$ If $COV_{VS} > 0.30$ the geotechnical engineer shall submit to the PCS/GDS either a recommended (with documentation) site stiffness, $\overline{\boldsymbol{V}}_s$, and Site Class to be used for the project or a request to perform a site-specific response analysis in accordance with Section 12.8.
3	The site stiffness, $\overline{V}_s$, is then computed as follows: $$\overline{\boldsymbol{V}}_s = \overline{\boldsymbol{V}}_s^{'} \qquad \text{Equation 12-33}$$ Use the new site stiffness, $\overline{V}_s$, to determine the Site Class.

When a project site has more than one Site Class due to soil spatial variations along the project alignment or when different structural components (bridge abutment, interior bents, embankments, etc.) require differing depth-to-motion, Z_{DTM}, the designer will need to evaluate the Site Class for each structure component being designed. Guidance in selecting the most appropriate Site Class for the structure being designed can be found in Section 12.4.3.

The steps for determining the project Site Class are described in Table 12-21.

Table 12-21, Site Class Determination Procedure

Step	Description
1	Check for the three criteria of Site Class F shown in Table 12-22 that would require a site-specific response evaluation. If the site meets any of these criteria, classify the project site as Site Class F.
2	Check for the existence of a soft soil layer with a total thickness, H > 10 ft (3 m). A soft soil layer is defined by: $PI > 20$, $w \geq 40\%$, and $\bar{s}_u < 500$ psf (25 kPa). If this criteria is satisfied, the project site is a Site Class E.
3	If a Site Class has not been assigned using Steps 1 and 2 above then compute the site stiffness, $\overline{\boldsymbol{V}}_{\boldsymbol{S}}$, using the procedures in Section 12.3.4 and Table 12-20.
4	Determine the Site Class based on the site stiffness, $\overline{V}_S$, using Table 12-22.

Table 12-22, Site Class Seismic Category

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 FT (30 M) Below Z_{DTM} SITE STIFFNESS $\overline{V}_s$
A	Hard Rock	$\overline{V}_s$ > 5,000 ft/sec ($\overline{V}_s$ >1500 m/sec)
B	Rock	2,500 < $\overline{V}_s$ ≤ 5,000 ft/sec (760 < $\overline{V}_s$ ≤ 1500 m/sec)
C	Very Dense Soil and Soft Rock	1,200 < $\overline{V}_s$ 2,500 ft/sec (360 < $\overline{V}_s$ ≤ 760 m/sec)
D	Stiff Soil	600 ≤ $\overline{V}_s$ ≤ 1,200 ft/sec (180 ≤ $\overline{V}_s$ ≤ 360 m/sec)
E	Soft Soil	$\overline{V}_s$ < 600 ft/sec ($\overline{V}_s$ < 180 m/sec)
		Any profile with more than 10 ft (3m) of soft clay defined as: *P*I > 20; w ≥ 40%; and $\overline{\tau} = \overline{s}_u$ < 500 psf (25 kPa)
F	Soils Requiring Site Specific Response Evaluation	Any soil profile containing one or more of the following characteristics: 1. Peats and/or highly organic clays (*H*>10 ft [3 m] of peat and/or highly organic clay where *H* = thickness of soil) 2. Very high plasticity clays (*H*>25 ft [8 m] with *PI* > 75) 3. Very thick soft/medium stiff clays (*H*> 120 ft [36 m])

Definitions:

PI = Plasticity Index (AASHTO T89, T90 or ASTM D 4318)

w = Moisture Content (AASHTO T265 or ASTM D 2216)

$\overline{V}_s$ = Average shear wave velocity for the upper 100 ft (30 m) below Z_{DTM}. (ft/sec or m/sec)

$\overline{\tau}$ = Average undrained shear strength ($\overline{\tau} = \overline{s}_u$) for cohesive soils in the upper 100 ft (30 m) below Z_{DTM}. (psf or kPa) (AASHTO T208 or T296 or ASTM D2166 or D2850)

Z_{DTM} = Depth-to-motion is the location where the ground motion transmits the ground shaking energy to the structure.

Notes:

(1) The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

(2) The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 feet (30m) below Z_{DTM}, surficial shear wave velocity measurements may be extrapolated to assess shear wave velocities.

(3) Site Classes A and B should not be used when there is more than 10 feet (3m) of soil between the rock surface and the depth-to-motion, Z_{DTM}. When rock is encountered within the 100 feet (30m) below the depth-to-motion, Z_{DTM}, and the soil layer is more than 10 feet (3m) use the Site Class pertaining to the soil above the rock.

(4) A Site Class F is not required if a determination is made that the presence of such soils will not result in a significantly higher response of a bridge. Consideration of the effects of depth-to-motion, Z_{DTM}, shall be taken into account when making this determination. Such a determination must be approved by the PCS/GDS.

12.4.2 Depth-To-Motion Effects On Site Class and Site Factors

The Site Class soil profile under evaluation should begin at the anticipated depth-of-motion, Z_{DTM}, for the structure being designed. The depth-to-motion, Z_{DTM}, is the location where the ground motion transmits the ground shaking energy to the structure being designed. When the depth-to-motion is identified, a structure specific Site Class is determined for the soil profile extending 100 feet (30 m) below the depth-to-motion, Z_{DTM}. Typical structures where a Site Class is needed are bridges, roadway embankments, earth retaining systems, and other roadway structures. The depth-to-motion, Z_{DTM}, can affect the Site Class significantly, particularly, for single component soil-structure interaction (SC-SSI) systems such as pile bents, where soft soils with a Site Class E are at the surface with underlying stiff soils with a Site Class D. If the depth-to-motion, Z_{DTM}, were located below the Site Class E soils, a Site Class D would be selected.

When structures are founded on shallow foundations, the depth-to-motion, Z_{DTM}, is typically located at the base of the structure, such as the base of an embankment fill, bottom of a footing, etc. The effects of fill overburden pressures (i.e. embankment fill) over the underlying soils should be included in the Site Class computations.

When structures are founded on deep foundations, the depth-to-motion determination is more complex because of the soil-structure interaction and should be evaluated jointly between the geotechnical engineer and structural engineer. The depth-to-motion, Z_{DTM}, location for deep foundations is at some point below the ground surface depending on the soil-structure components and their horizontal stiffness. Soil-structure interaction can be characterized as either a single component (i.e. soil-pile interaction) or a multi-component (i.e. soil-pile-footing and soil-pile).

A single component soil-structure interaction (SC-SSI) would be a bridge interior bent supported on a spread footing, bridge interior bent supported by a bent cap above the ground and piles or drilled shafts embedded in the ground, or a single bridge column supported by a drilled shaft. The depth-to-motion, Z_{DTM}, for the spread footing case would be the bottom of the footing. The depth-to-motion, Z_{DTM}, for the pile or drilled shaft bridge foundations listed can be estimated as the point-of-fixity typically used by structural engineers in their structural evaluations. The buckling point-of-fixity is typically used for preliminary analyses. The point-of-fixity is the point at which the earth pressures adequately resist a couple created by the moment, resist the lateral shear, or both.

A multi-component soil-structure interaction (MC-SSI) is comprised of various soil-structure system components with each component having different horizontal stiffness as illustrated in Figure 12-29. Figure 12-29 illustrates an embedded pile group footing where the soil-pile-footing system component has a horizontal stiffness and the soil-pile system component has another horizontal stiffness. If the soil-pile-footing horizontal stiffness were considerably greater than the pile group-soil horizontal stiffness as illustrated in Figure 12-29(A), the depth-to-motion, Z_{DTM}, would be at the base of the footing. Conversely, if the soil-pile system component stiffness were greater than the stiffness of the soil-pile-footing system as illustrated in Figure 12-29(B), the depth-to-motion, Z_{DTM}, would be located at some depth below the surface similar to the SC-SSI. The depth-to-motion, Z_{DTM}, for the soil-pile system will need to account for the pile group interaction.

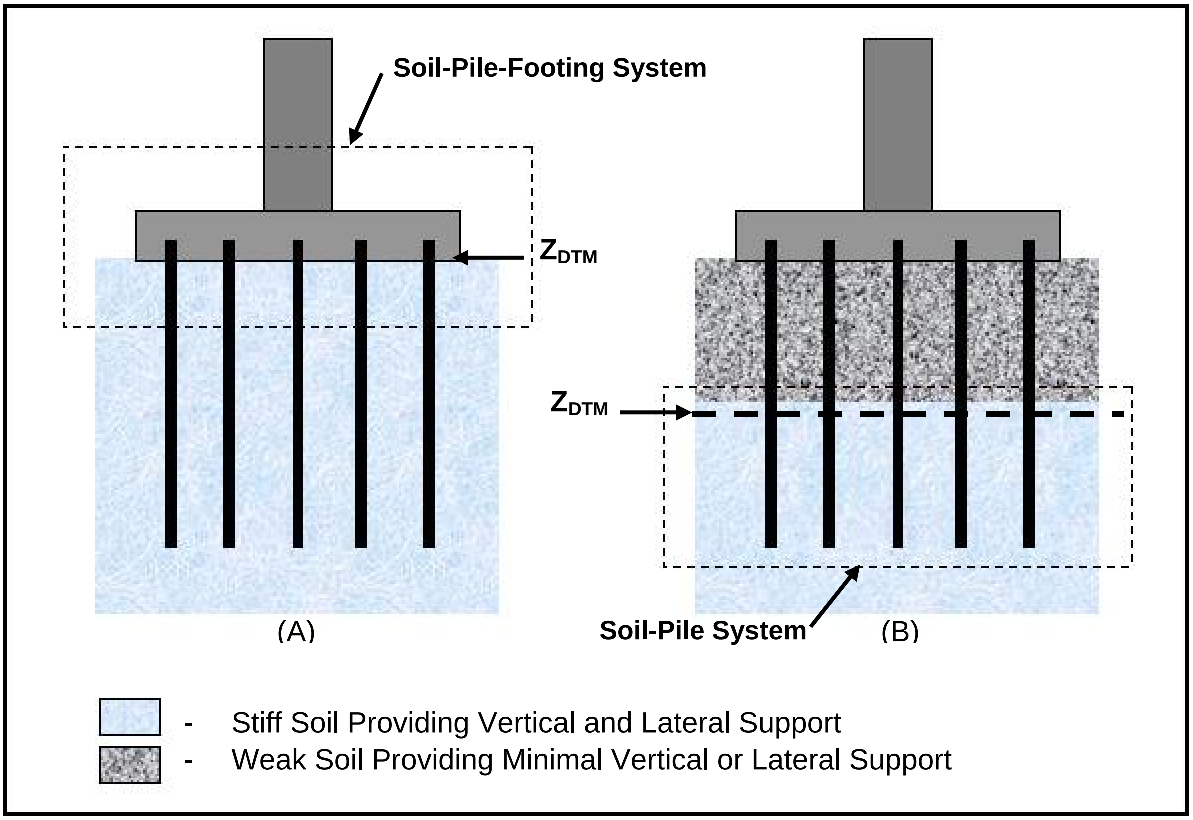


Figure 12-29, Multi-Component Soil-Structure Interaction (MC-SSI)

12.4.3 Site Class Variation Along a Project Site

The procedures for determining Site Class works well when relatively uniform soil conditions are encountered at a project site. As has been seen the depth-to-motion concept discussed in Section 12.4.2 can produce various Site Classes depending on the type of structure or component being analyzed. Using a single Site Class for designing individual structures (bridges, roadway embankments, retaining walls, and miscellaneous roadway structures) can be accomplished by evaluating the primary mechanism by which energy is transferred from the ground to the structure.

If the Site Class varies between the interior bents and abutments of a bridge, the design Site Class of the bridge structure must be evaluated jointly between the geotechnical engineer and the structural engineer. The motion at the bridge abutment for short bridges with relatively few spans will generally be the primary mechanism by which energy is transferred to the bridge superstructure and therefore the Site Class at the bridge abutment would govern. The Site Class for bridges may differ significantly along the bridge alignment due to variability in soil conditions such as one abutment is founded on rock (Site Class B), the other abutment is founded on soft soils (Site Class E), and the interior bents are founded on stiff soils (Site Class D). In this circumstance, the primary mechanism by which energy is transferred to the bridge is more difficult to determine. If only a single site response will be used in the analyses, then an envelope could be developed that captures the predominant periods for the entire spectrum using the various site classes. If the structural analytical method allows the input of several motions at different locations, then several Site Classes should be used.

The geotechnical engineer is responsible for evaluating soil conditions and the extent of site variability (if any) at the bridge location and then determining the Site Class for each individual soil region based on the guidelines provided in this Section. The geotechnical engineer and the structural engineer will then jointly evaluate the appropriate Site Class to be used for the structural design of the bridge.

12.4.4 South Carolina Reference Site Classes

A Site Class was computed for the USGS Shear Wave Velocity Data and SCEMD Seismic Risk and Vulnerability Study based on the shear wave reference profiles in Sections 12.3.3.1 and 12.3.3.2, respectively. The reference Site Class was determined for each shear wave profile using a site stiffness ($\bar{V}_s$) computed in accordance with 12.3.4 for a depth-to-motion at the ground surface (Z_{DTM} = 0).

The site stiffness and corresponding Site Class for the USGS Shear Wave Velocity Data are provided in Table 12-23.

Table 12-23, USGS Site Stiffness and Site Class (Modified Odum et al., 2003)

Site No.	Site Name	Latitude (degrees)	Longitude (degrees)	Surficial Geology (1)	Site Stiffness $\bar{V}_S$ (m/s)	Site Stiffness $\bar{V}_S$ (ft/sec)	Site Class (2, 3)
1	Lake Murray Spillway	35.052	81.210	Fill, Pz	661	2,168	C
2	Fort Jackson	34.028	90.912	K_u	465	1,525	C
3	Deep Creek School	33.699	79.351	Q?, K_u	246	807	D
4	Black Mingo	33.551	79.933	Q, T_l	477	1,565	C
5	Santee Ls	33.235	80.433	T_l	583	1,912	C
6	The Citadel, Charleston	32.798	79.958	Q, T_u	248	813	D
7	US Hwy. 17, Charleston	32.785	79.955	Fill, Q	182	597	E
8	Isle of Palms	32.795	79.775	Q_h, T_u	179	587	E
9	USNSN	33.106	80.178	Q, T_u	464	1,521	C

(1) Definitions: Q – Quaternary; T_u – upper Tertiary; T_l – lower Tertiary; K_u – upper Cretaceous; P_z - Paleozoic

(2) Site Classes were evaluated based on Table 12-22 using the shear wave velocities in ft/sec.

(3) The depth-to-motion (Z_{DTM} = 0) for the reference Site Class computations was assumed to be the ground surface. Selection of a depth-to-motion below the surface (Z_{DTM} > 0) could significantly affect the Site Class determination.

The site stiffness and corresponding Site Class for the SCEMD Seismic Risk and Vulnerability Study are provided in Table 12-24.

Table 12-24, USGS Site Stiffness and Site Class (Modified URS Corporation, 2003)

Site [1] No.	Site Response Category [1]	Geology	Site Stiffness $\overline{V}_S$ (m/s)	Site Stiffness $\overline{V}_S$ (ft/sec)	Site Class [2, 3]
1, 2, 4 [4]	Piedmont/Blue Ridge, Savannah River, Myrtle Beach [4]	Crystalline	3,400	11,152	A
1	Piedmont/Blue Ridge	Piedmont/Blue Ridge	453	1,486	C
2	Savannah River	Savannah River	355	1,165	D
3	Charleston	Charleston	328	1,077	D
4	Myrtle Beach	Myrtle Beach	239	784	D

(1) Site Response Categories are shown in Figure 12-10.
(2) Site Classes were evaluated based on Table 12-22 using the shear wave velocities in ft/sec.
(3) The depth-to-motion (Z_{DTM} = 0) for the reference Site Class computations was assumed to be the ground surface. Selection of a depth-to-motion below the surface (Z_{DTM} > 0) could significantly affect the Site Class determination.
(4) Various Site Nos. and Site Response Categories are provided for a crystalline geology to account for transition zones between geologies and to allow for any hard-rock basement outcrops located outside of the Piedmont/Blue Ridge Response Category.

12.5 SC EARTHQUAKE HAZARD ANALYSIS

The SC Seismic hazard maps shall be used for all “Typical SCDOT Bridges” as defined by Sections 1.4 and 1.5 of the SCDOT *Seismic Design Specifications for Highway Bridges*. For non-typical bridges, the PCS/GDS will specify and/or approve appropriate geotechnical earthquake engineering provisions on a project specific basis. The SC Seismic Hazard maps are described in Section 11.9.2, Probabilistic Earthquake Hazard Maps. The seismic hazard information generated from these maps includes the PGA and PSA for 0.5Hz, 1.0Hz, 2.0Hz, 3.3Hz, 5Hz, 6.7Hz, and 13Hz frequencies for the FEE and SEE design earthquakes at hard rock basement outcrop or at geologically realistic site condition.

12.6 ACCELERATION RESPONSE SPECTRUM

The acceleration response spectrum of a specific earthquake motion is a plot of the maximum spectral acceleration, S_a, response of a series of linear single degree-of-freedom systems with the same damping and mass, but variable stiffness. The South Carolina Seismic Hazard maps generate a probabilistic Uniform Seismic Hazard of PGA and PSA at either a hard-rock basement outcrop or at a geologically realistic site conditions (i.e. B-C Boundary in the Coastal Plain). The response spectrum at these locations needs to be adjusted for the local site effects. The local site effects are influenced by the soil stiffness (resonant frequency) of the soil column above the location where ground motion was generated. The soil column extends to the location where the ground motion transmits the ground shaking energy to the structure being designed, also referred to as the depth-to-motion, Z_{DTM}, in Section 12.4.2.

The maximum local site amplification occurs when the predominant or maximum period, T_{max}, of the rock outcrop ground motion, the soil deposit's natural period, T_N, and the fundamental period of the structure, T_s, are all in phase. The relationship between rock outcrop and soil surface motions is complex and depends on numerous factors including the fundamental period of the soil profile, strain dependency of soil stiffness and damping, and the characteristics of the rock outcrop motion (Seed and Idriss, 1982).

The effects of local soil site conditions such as rock outcrop, stiff site conditions, soft to medium clay and sand, and deep cohesionless soils on the response spectra shapes (5% damped) are shown in Figure 12-30 (Seed et, al., 1976). Normalized spectral shapes were computed by dividing the spectral acceleration by the peak ground acceleration (PGA) at the surface. These spectral shapes were computed from motion records made on rock and soil sites at close distances to earthquakes ($6 \leq M_w \leq 7$). These normalized spectral curves show that spectral response amplification is significantly greater at longer periods ($\approx$ 1 second) with soil site conditions that have decreasing soil site stiffness. The observed variations in spectral response as a function of subsurface site conditions underscore the importance of properly evaluating the project Site Class in accordance with Section 12.4.

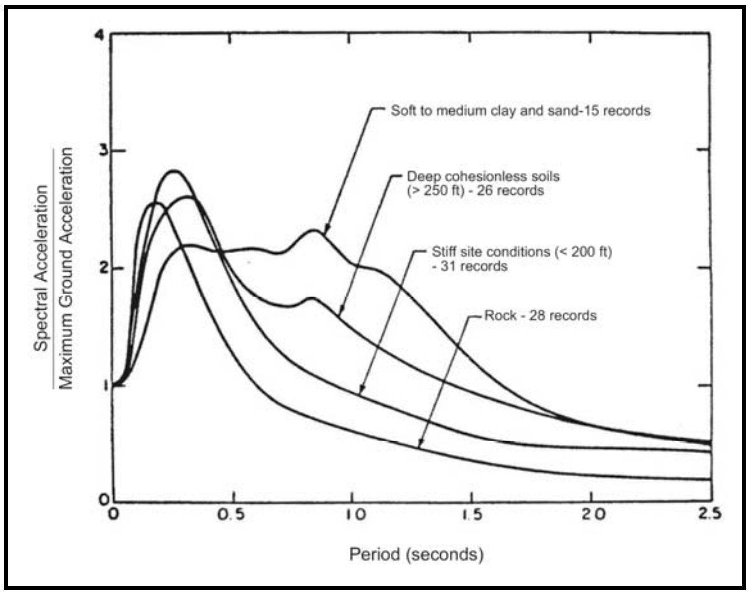


Figure 12-30, Soil Site Effects on Average Normalized Response Spectra (Seed et al., 1976)

Amplification of peak accelerations from the Uniform Hazard occurs when the resonant frequency, f_o, of the soil deposit is close to the predominant frequency or maximum period, T_{max}, of ground motions at either the B-C Boundary or hard-rock basement outcrop.

The natural period, T_N, of the site can be estimated by the following equation.

$$\boldsymbol{T_N} = \frac{1}{\boldsymbol{f_o}} = \frac{4\boldsymbol{H}}{\boldsymbol{V^*_{s,H}}} \qquad \text{Equation 12-34}$$

Where,

f_o = resonant frequency of the soil deposit thickness (H). Units Hz

$V^*_{S,H}$ = equivalent uniform soil profile stiffness of thickness (H). Units ft/sec (Section 12.3.4)

H = thickness of soil deposit above B-C Boundary or hard-rock basement outcrop depending on the level where ground motion input has been developed. Units feet

As can be seen by Equation 12-34, the natural period of the site (T_N) is influenced by the site equivalent uniform soil profile stiffness and the thickness of the soil deposit (H). A general trend is observed in Figure 12-31 that the natural period of a site (T_N) increases as the site stiffness decreases while keeping the soil deposit thickness the same. In addition, as the thickness of the soil profile increases (keeping the site stiffness the same), the natural period of the site increases again. Consequently, a combination of lower site stiffness and increased soil deposit thickness will work together to increase the natural period of the site. At the same time, a reduction in the natural period of the site is observed primarily when the equivalent uniform soil profile stiffness ($V^*_{S,H}$) increases as the depth of the soil profile decreases.

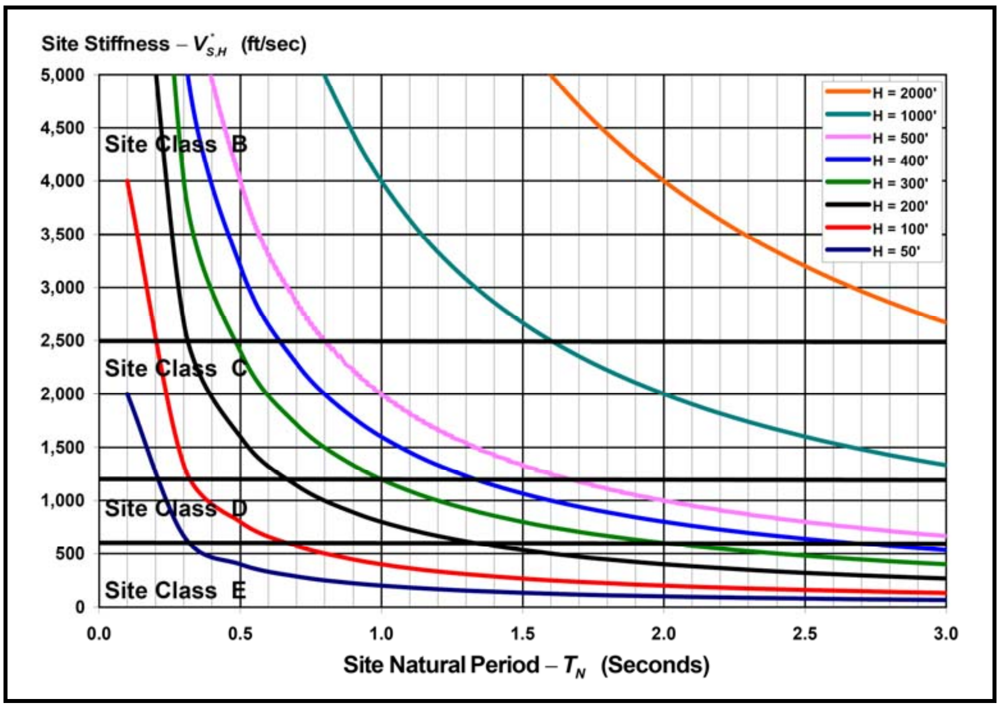


Figure 12-31, Site Natural Period (T_N)

A recent study by Green (2001) reveals that the maximum period, T_{max}, of the bedrock motion in the Central and Eastern United States (CEUS) varies $0.05 < T_{max} < 0.10$ sec. as compared to the Western United States (WUS) which varies $0.15 < T_{max} < 0.25$ sec. This would indicate that South Carolina sites with low natural periods, T_N, in the range of 0.075 seconds would be subject to greater amplification.

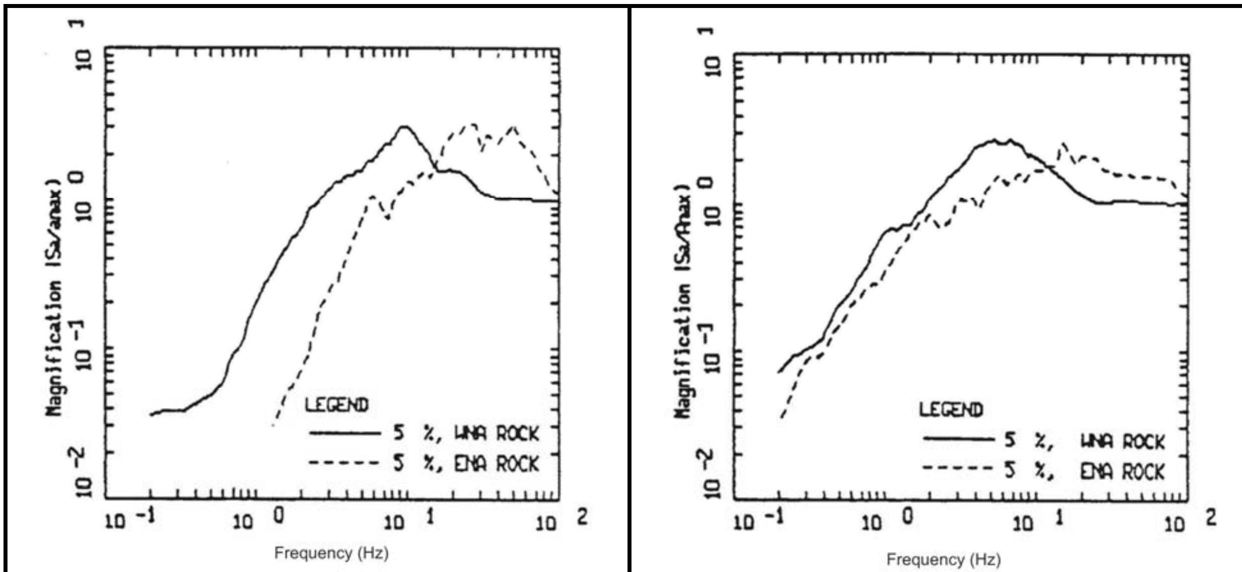
It is equally important to know the fundamental period of the structure (i.e. bridge, earth retaining structure, dam, etc.) being designed since structures with periods similar to the period of the ground motion reaching the structure will tend to exert higher seismic loads (demand) and potentially cause significant damage to the structure.

The local site effects are taken into account by performing a site response analysis using the SC Seismic Hazard Maps (Section 12.7) or by performing a site-specific response analysis (Section 12.8).

The following subsections 12.6.1, 12.6.2, and 12.6.3 describe special site conditions that may influence the site response that typically cannot be addressed by simplified response methods that use the SC Seismic Hazard Maps (Section 12.7).

12.6.1 Effects of Rock Stiffness WNA vs. ENA

The effects of rock stiffness (shear wave velocity) and damping on normalized response spectra shapes (5% damped) on rock sites are shown in Figure 12-32 (Silva and Darragh, 1995). Normalized spectral shapes were computed by dividing the spectral acceleration by the peak ground acceleration (PGA) at the surface. Normalized response spectra were computed for Western North America (WNA), representative of soft rock encountered in California and for Eastern North America (ENA), representative of hard rock encountered in the Eastern United States. The normalized response spectra were computed from motion records made on rock sites at close distances to earthquakes (M_w = 4.0 and 6.4). These normalized spectral curves show that ENA spectral response amplification is greater at longer periods when compared to WNA spectral response. This effect of higher amplification at longer periods is more evident for smaller earthquakes because of higher corner frequencies for smaller magnitude earthquakes (Boore, 1983; Silva and Green, 1989; Silva and Darragh, 1995).



Earthquake M_w ~ 4.5	Earthquake M_w ~ 6.5

Figure 12-32, WNA / ENA Rock Effects on Normalized Response Spectra (Silva and Darragh, 1995)

12.6.2 Effects of Weathered Rock Zones Near the Ground Surface

Some caution should be exercised when evaluating the site response of sites where weathered rock zones are near the surface such as in the Blue Ridge/Piedmont Units and in transition areas between the Piedmont Unit and the Coastal Plain Unit. Transition areas between physiographic units can be found in the Columbia, SC metropolitan area. The Columbia, SC area generally consists of 10 to 30 feet of surficial soils ($200 \leq V_s \leq 500$ ft/sec), underlain by 30 to 90 feet of a weathered rock zone ($2{,}500 < V_s < 8{,}000$ ft/sec), followed by a hard-rock basement outcrop (V_s >11,000 ft/sec). A recent site-specific response study (Chapman, 2008) of the Columbia, SC area compared spectral accelerations modeled at a B-C boundary (weathered rock) outcropping conditions and hard-rock outcropping conditions with a weathered rock zone modeled by a shear wave velocity gradient from 2,500 to 8,000 ft/sec on 1.5 ft. increments. This study found that the spectral accelerations for the two models were similar for frequencies up to 10 Hz. (periods > 0.10 seconds). The spectral accelerations increased for frequency greater than 10 Hz. (periods < 0.10 seconds) for the model with hard-rock outcropping conditions and a velocity graded weathered rock zone. The magnitude of the increase in spectral acceleration was dependent on the thickness of the graded weathered rock zone.

Based on this study (Chapman, 2008) the following preliminary guidelines are provided:

1. **Coastal Plain Unit with sedimentary surface soils:** When ground motions are generated using a geologically realistic site condition using Senario_PC (2006) the thickness of the firm Coastal Plain sediment and/or weathered rock zone will be modeled approximately by the transfer function that places the ground motion at the B-C boundary (V_s = 2,500 ft/sec) and therefore the amplification observed from weathered rock thickness greater than 30 feet will not be as significant.

2. **Blue Ridge/Piemont Unit with Weathered Rock Zone:** The Three-Point site response method can only be used if the weathered rock thickness ($2{,}500 \leq Vs \leq 8{,}000$ ft/sec) is less than 30 feet thick. When performing site-specific response analyses in the Blue Ridge/Piedmont units with weathered rock zone ($2{,}500 \leq Vs \leq 8{,}000$ ft/sec) thickness greater than 30 feet, this zone must be modeled by a shear wave velocity gradient. If the thickness (d_{WR}) of the weathered rock zone is unknown, a sensitivity analysis of the thickness will be required to determine the amplification effects on the spectral accelerations and PGA.

12.6.3 Effects of Soil Softening and Liquefaction on Spectral Acceleration

Youd and Carter (2005) have studied the effects of soil softening and liquefaction on spectral accelerations of five instrumented sites. Three of the sites were in the United States (California) and the other two in Japan. Youd and Carter (2005) made the following observations:

1. Soil softening due to increased pore water pressure generally reduces short period spectral accelerations (T < 1.0 sec) as compared to those spectral accelerations that would have occurred without soil softening.

2. Soil softening may have little influence on short period spectral accelerations (T < 1.0 sec) when soil softening occurs late in the strong motion sequence.

3. Soil softening usually amplifies or enhances long period spectral accelerations (T > 1.0 sec) due to lengthening of the natural period of the site as it softens (See Figure 12-31). When liquefaction-induced ground oscillations continue after earthquake shaking, there may be considerable enhancement of the long-period (T > 1.0 sec) spectral accelerations.

When a site-specific response analysis is not performed and the simplified response methods that use the SC Seismic Hazard Maps (Section 12.7) are used, the effects of soil softening and liquefaction on the design spectral response generated will have the following implications to the structures being designed.

1. For structures with short-fundamental periods (T < 1.0 sec), the design spectral accelerations will conservatively envelope the actual spectral acceleration for sites where soil softening or liquefaction occurs early in the strong motion sequence.

2. For structures with long-fundamental periods (T > 1.0 sec), the design spectral accelerations may be <u>unconservative</u> due to the lengthening of the natural period of the site. For these types of structures with long-fundamental periods (T > 1.0 sec), a site-specific response analysis should be considered.

12.6.4 <u>Horizontal Ground Motion Response Spectra</u>

The SCDOT *Seismic Bridge Design Specifications* requires safety and functional evaluations for bridges based on the bridge Operational Classification, OC. All bridges (OC = I, II, or III) require a structural response evaluation using the Safety Evaluation Earthquake (SEE). Bridges with an OC = I or II also require a structural evaluation using the Functional Evaluation Earthquake (FEE) only if the project site has the potential for liquefaction or slope instability at bridge abutments and no geotechnical mitigation is performed.

The horizontal acceleration design response spectrum (ADRS) curves can be determined by either the Three-Point method (Section 12.7) or the Site-Specific response analysis (Section 12.8) using the selection criteria in Table 12-25.

Table 12-25, Site Response Selection Criteria

Physiographic Unit [1]	Site Response Method	Site Class	Weathered Rock Thickness d_{WR} [2] (feet)	Site Condition	Outcrop Description	Comments
Coastal Plain Unit	Three-Point	A, B, C, D, E	d_{WR} Any	Geologically Realistic	V_S = 2,500 ft/sec B-C Boundary	Site Class should be based on soil column consisting of sediment soils [2] (V_s < 2,500 ft/sec). Document Site Stiffness selection (Table 12-22).
	Site-Specific Response	F or Required by SCDOT	d_{WR} Any	Geologically Realistic	V_S = 2,500 ft/sec B-C Boundary	Soil column model must extend to hypothetical firm Coastal Plain outcrop equivalent of B-C boundary (V_s >2,500 ft/sect). Document soil column properties and soil stratification sensitivity.
				Hard-Rock Basement Outcrop	V_S = 11,500 ft/sec Top of "A" Boundary	Use of ground motions generated at the hard-rock basement outcrop will require written permission by the PCS/GDS. The soil column model must extend to hard-rock basement outcrop (V_s>11,500 ft/sec). The soil column development must be documented thoroughly and extensive soil stratification sensitivity analyses must be performed, particularly below the B-C boundary.
Outside Coastal Plain Unit	Three-Point	A, B, C, D, E	0 < d_{WR} < 30	Geologically Realistic	V_S = 8,200 ft/sec Top of "A" Boundary	Site Class should be based on soil column consisting of sediment soils [2] (V_s < 2,500 ft/sec). Document Site Stiffness selection (Table 12-22). Site Class A must not be used. Select Site Class B only if depth-to-motion (Z_{DTM}) is at top of weathered rock zone (V_s > 2,500 ft/sec)
			d_{WR} = 0	Hard-Rock Basement Outcrop	V_S = 11,500 ft/sec Top of "A" Boundary	Select Site Class A only if depth-to-motion (Z_{DTM}) is at top of hard-rock zone (V_s > 11,000 ft/sec). Note that hard-rock must be verified by shear wave velocity measurements of hard-rock (V_s >11,000 ft/sec).
	Site Specific Response	F or Required by SCDOT	d_{WR} ≤ 30	Geologically Realistic	V_S = 8,200 ft/sec Top of "A" Boundary	Soil column model must extend to hypothetical firm Coastal Plain outcrop equivalent of a hypothetical outcrop of Piedmont weathered rock (Vs >8,000 ft/sect). Document soil column development and soil stratification sensitivity.
			d_{WR} > 30	Hard-Rock Basement Outcrop	V_S = 11,500 ft/sec Top of "A" Boundary	Soil column model must extend to hard-rock basement outcrop (Vs>11,000 ft/sec). Document soil column development and soil stratification sensitivity. The weathered rock zone (2,500 ≤ V_s ≤ 11,500 ft/sec) must be modeled by a shear wave velocity gradient. If thickness (d_{WR} > 30 ft.) of the weathered rock zone is unknown, a sensitivity analysis of the thickness will be required Select Site Class B only if depth-to-motion (Z_{DTM}) is at top of weathered rock zone (V_s > 2,500 ft/sec)

[1] If Senario_PC (2006) indicates a zero sediment thickness (d_S = 0) the site is assumed to be outside of the Coastal Plain (Blue Ridge/Piedmont). If the sediment thickness is greater than zero (d_S > 0) the site is assumed to be in the Coastal Plain.

[2] Weathered rock zone with shear wave velocities 2,500 – 8,000 ft/sec.

Horizontal acceleration design response spectrum (ADRS) curves described in Sections 12.7 and 12.8 are generated for the design earthquakes (SEE and/or FEE) as needed for the structural engineer to perform a structural evaluation. The horizontal ADRS curves are supplied to the structural engineer in the form of a curve and tabulated values of spectral accelerations, S_a, in units of gravity (g) and corresponding time period, T, in units of seconds.

12.6.5 Vertical Ground Motion Response Spectra

Recent studies shown in Figure 12-33 reveal that the ratio of vertical to horizontal ground motion response spectra can vary substantially from the nominal two-thirds (2/3) ratio commonly used. Studies show that the two-thirds ratio of vertical to horizontal ground motion response spectra may be conservative for periods of vibration longer than 0.2 seconds. For periods of vibration shorter than 0.2 seconds the ratio of vertical to horizontal ground motion response spectra may exceed the two-thirds value and may be on the order of 1 to 1.5 times the horizontal for earthquakes with close source-to-site distances and periods of vibration of less than 0.1 seconds. Although the studies shown in Figure 12-33 are from ground motion data from the western United States (WUS), Chiou et al. (2002) indicates that the ratios for the Central and Eastern United States (CEUS) are not greatly different from the ratios in the WUS.

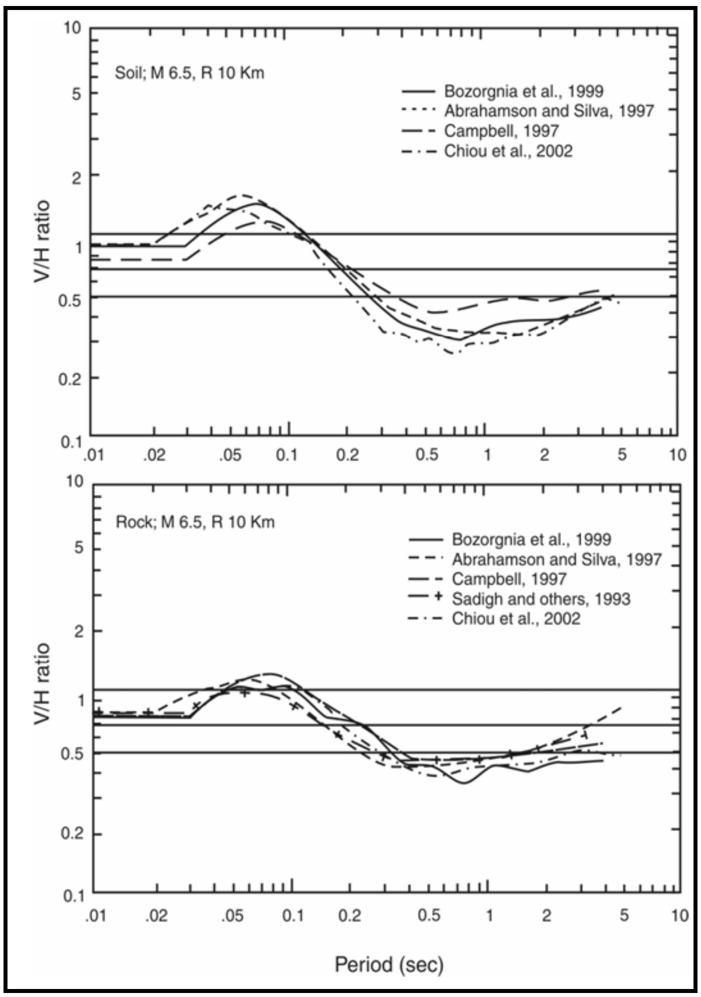


Figure 12-33, Vertical/Horizontal Spectral Ratios vs. Period (Buckle et al, 2005)

Because there are currently no accepted procedures for constructing the vertical response spectra or having an appropriate relationship with the horizontal response spectra constructed using the SC Seismic Hazard maps, Section 12.7, the two-thirds ratio of vertical-to-horizontal response spectra shall be used for bridges with natural periods of vibration of 0.2 seconds or longer. When the bridge's natural period of vibration is less than 0.2 seconds, a site-specific vertical response spectra using the results of recent studies such as those shown in Figure 12-33 should be used to develop the vertical ground motion response spectra.

12.7 SC SEISMIC HAZARD MAPS SITE RESPONSE ANALYSIS

12.7.1 ADRS Curves for FEE and SEE

As described in Section 12.6.2 there are two design earthquakes that are used for evaluation of SCDOT structures, the Functional Evaluation Earthquake (FEE) and the Safety Evaluation Earthquake (SEE). The PGA and spectral response accelerations used in Section 12.7.2 will depend on which design earthquake is being analyzed.

The horizontal ADRS curves generated using the SC Seismic Hazard maps will be based on a 5% viscous damping ratio because the pseudo spectral accelerations (PSA) obtained from the SC Seismic Hazard maps have been generated for 5% damping.

12.7.2 Local Site Effects on PGA

The peak ground acceleration at the existing ground surface is determined by evaluating the local site effects on the mapped peak ground acceleration at the B-C boundary, $PGA_{B\text{-}C}$. The $PGA_{B\text{-}C}$ shall be obtained for the appropriate design earthquake (FEE or SEE) being analyzed. The $PGA_{B\text{-}C}$ value shall be generated from the SC Seismic Hazard maps as indicated in Sections 12.5 and 11.9.2 at the B-C boundary. The *PGA* shall be determined by adjusting the $PGA_{B\text{-}C}$ based on Site Class using the following equation.

$$PGA = F_{PGA} \cdot PGA_{B-C} \qquad \text{Equation 12-35}$$

Where:

PGA	=	peak ground acceleration at the existing ground surface (period, T = 0.0 sec.) adjusted for local site conditions
$PGA_{B\text{-}C}$	=	mapped peak ground acceleration at the B-C boundary (period, T = 0.0 sec.)
F_{PGA}	=	site coefficient defined in Table 12-26, based on the Site Class and the mapped peak ground acceleration, $PGA_{B\text{-}C}$.

Table 12-26, F_{PGA} Site Factor for Peak Ground Acceleration (PGA)

Site Class	Mapped Peak Ground Acceleration (Period, T = 0 sec.), $PGA_{B\text{-}C}$ (1)				
	$PGA_{B\text{-}C} \leq 0.10$	$PGA_{B\text{-}C} = 0.20$	$PGA_{B\text{-}C} = 0.30$	$PGA_{B\text{-}C} = 0.40$	$PGA_{B\text{-}C} \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F(2)	N/A	N/A	N/A	N/A	N/A

Notes:
(1) Use linear interpolation for intermediate values of *PGA*.
(2) Site-specific response analysis shall be performed.

12.7.3 Local Site Effects on Spectral Response Accelerations

The design spectral response accelerations for short period (T = 0.20 second), S_{DS}, and the long period (T = 1.0 second), S_{D1}, at the ground surface are determined by evaluating the local site effects on the horizontal spectral response accelerations for short period (0.20 second), S_S, and long period (1.0 second), S_1, at the B-C boundary. The horizontal spectral accelerations S_s and S_1 values shall be obtained for the appropriate design earthquake (FEE or SEE) being analyzed. The S_s and S_1 values are generated from the SC Seismic Hazard maps as shown in Sections 12.5 and 11.9.2 at the B-C boundary (geologically realistic). Design spectral response accelerations S_{DS} and S_{S1} shall be determined using Equation 12-36 and Equation 12-40, respectively.

$$S_{DS} = F_a S_S \qquad \text{Equation 12-36}$$

$$S_{D1} = F_v S_1 \qquad \text{Equation 12-37}$$

Where:

- S_{DS} = design short-period (0.2-second) spectral response acceleration parameter
- S_{D1} = design long-period (1.0 second) spectral response acceleration parameter
- F_a = site coefficient defined in Table 12-27, based on the Site Class and the mapped spectral acceleration for the short-period, S_S.
- F_v = site coefficient defined in Table 12-28, based on the Site Class and the mapped spectral acceleration for the long-period, S_1.
- S_S = the mapped spectral acceleration for the short-period (0.2-second) as determined in Sections 12.5 and 11.8.2 at the B-C boundary
- S_1 = the mapped spectral acceleration for the one second period as determined in Sections 12.5 and 11.8.2 at the B-C boundary

Table 12-27, F_a Site Factor for Short-Period (0.2 sec = 5 Hz)

Site Class	Mapped Spectral Acceleration at Short-Periods (0.2 sec), S_s [1]				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F [2]	N/A	N/A	N/A	N/A	N/A

Notes:
(1) Use linear interpolation for intermediate values of S_S.
(2) Site-specific response analysis shall be performed.

Table 12-28, F_v Site Factor for Long-Period (1.0 sec = 1 Hz)

Site Class	Mapped Spectral Acceleration at Long-Period (1.0 sec), S_1 [1]				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F [2]	N/A	N/A	N/A	N/A	N/A

Notes:
(1) Use linear interpolation for intermediate values of S_1.
(2) Site-specific response analysis shall be performed.

12.7.4 Three-Point Acceleration Design Response Spectrum

The Three-Point method of constructing the horizontal ADRS curve is typically used for structures having natural periods of vibration between 0.2 second and 3.0 second. The Three-Point method has been shown by Power et al. (1997, 1998) to be unconservative in the CEUS for periods between 1.0 second and 3.0 seconds, and a Site Class B (Rock). When the fundamental period of the structure is less than 0.2 seconds or greater than 3.0 seconds, a site-specific response analysis as described in Section 12.8 may be required. The Multi-Point methods shall be used to evaluate the reasonableness of the Three-Point ADRS Curve as discussed in Section 12.7.5. Guidelines for constructing the Three-Point ADRS Curve are illustrated in Figure 12-34 and step-by-step instructions are provided in Table 12-29.

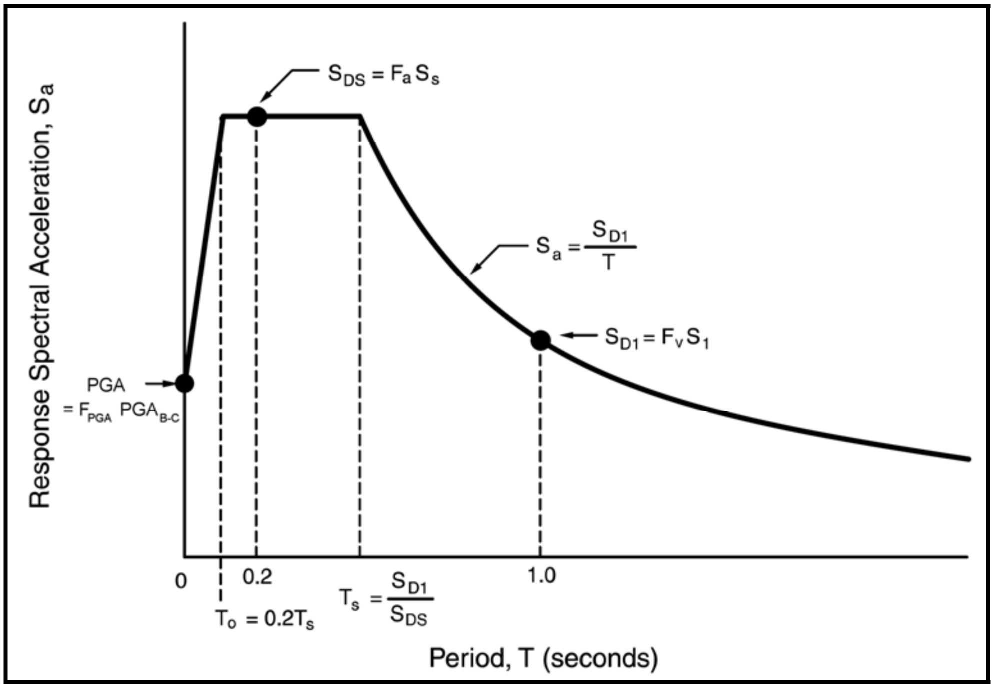


Figure 12-34, Three-Point ADRS Curve

Table 12-29, Three-Point ADRS Construction Procedures

Step	Procedure Description
1	The design short-period acceleration, S_{DS}, at period, T = 0.2 second is computed by using Equation 12-36 from Section 12.7.3. The design long-period acceleration, S_{D1}, at period, T = 1.0 second is computed by using Equation 12-37 from Section 12.7.3. $$S_{DS} = F_a S_S$$ Equation 12-36 $$S_{D1} = F_v S_1$$ Equation 12-37 Where values of F_a, F_v, S_s, and S_1 are obtained as indicated in Section 12.7.3.
2	Period markers T_o and T_s used in constructing the ADRS curves are defined by the following equations. $$T_o = 0.20T_s$$ Equation 12-38 $$T_s = \frac{S_{D1}}{S_{DS}}$$ Equation 12-39 Where S_{DS} and S_{D1} are obtained in Step 1.
3	The PGA at the existing ground surface at period, T=0.0 second is computed by using Equation 12-35 from Section 12.7.2. $$PGA = F_{PGA} \cdot PGA_{B-C}$$ Equation 12-35 Where F_{PGA} and PGA_{B-C} are obtained as indicated in Section 12.7.2.
4	The design spectral response acceleration S_a for periods, $T \leq T_o$, is computed by the following equation. $$S_a = PGA + \left[\left(S_{DS} - PGA \right) \left(\frac{T}{T_o} \right) \right]$$ Equation 12-40 Where, S_{DS} is obtained in Step 1, T_o is obtained in Step 2, and PGA is obtained in Step 3.
6	The design spectral response acceleration, S_a, for periods, $T_o \leq T \leq T_s$, is taken equal to S_{DS}, as obtained in Step 1.
7	The design spectral response acceleration, S_a, for periods, $T_s > T \leq 3.0$ seconds, is computed by the following equation. Equation 12-41 $$S_a = \frac{S_{D1}}{T}$$ Where, S_{D1} is obtained in Step 1.

12.7.5 Multi-Point Acceleration Design Response Spectrum

The Multi-Point method of constructing an ADRS curve shall be used to check the reasonableness of the Three-Point ADRS curve. This is accomplished by first constructing the Three-Point ADRS curve and then overlaying on the same graph the Multi-Point ADRS values as shown in Figure 12-35. The designer should be aware that Power and Chiou (2000) have found that the Multi-Point method may give ambiguous results for structures on sites other than rock (Site Class B). This is due to the Multi-Point method using the short period (0.2 seconds) site factor F_a for all the PSA values with periods less than or equal to 0.2 seconds and using long-period (1.0 seconds) site factor, F_v, for all periods greater than or equal to 1.0 seconds to compute the acceleration response spectrum. The Multi-Point method has been found to be appropriate for structures located on rock (Site Class B) because the site factors (F_a and F_v) for Site Class B are all unity, therefore no amplification or damping. Since the Multi-Point method is only used to check the reasonableness of the Three-Point ADRS curve, the construction of the Multi-Point ADRS Curve for Site Classes other than "B" should be adequate. Guidelines for constructing the Multi-Point ADRS curve are provided in Table 12-30.

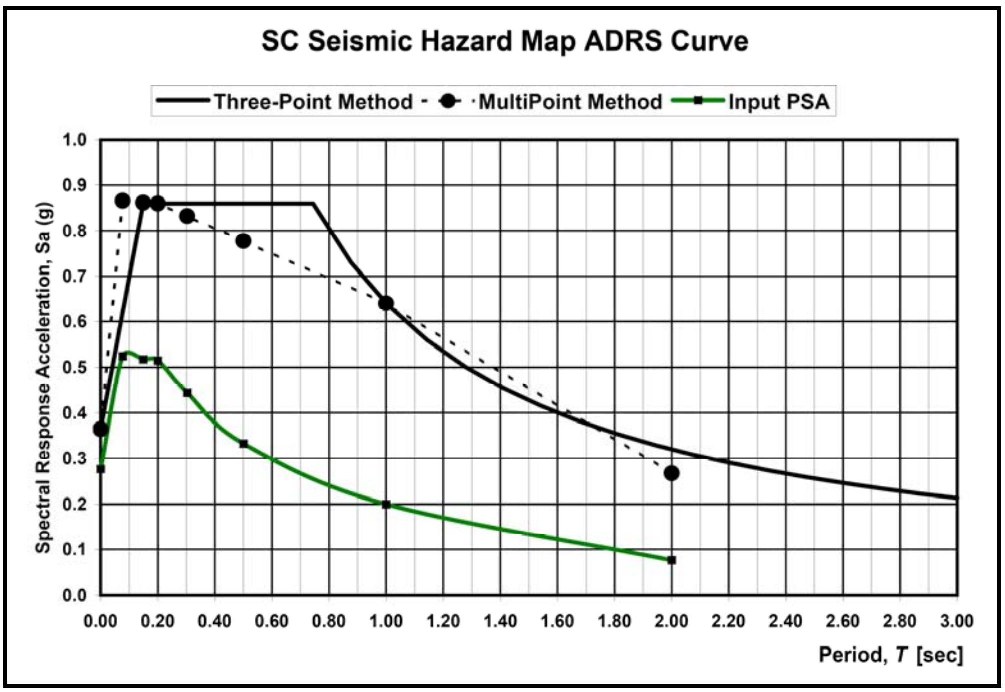


Figure 12-35, Three-Point/Multi-Point ADRS (Site Class=C)

Table 12-30, Multi-Point ADRS Construction Procedure

Step	Procedure Description
1	The FEE or SEE mapped pseudo spectral accelerations (PSA) for periods, T = 2.0 sec (0.5Hz), 1.0 sec (1.0Hz), 0.20 sec (5Hz), 0.15 sec (6.7Hz), 0.08 sec (13Hz) and PGA ($PGA_{B\text{-}C}$) are obtained from the SC Seismic Hazard map as indicated in Sections 12.5 and 11.9.
2	The PGA at the existing ground surface (Period, T=0) is computed by using Equation 12-35 from Section 12.7.2. $$PGA = F_{PGA} \cdot PGA_{B-C}$$ Equation 12-35 Where F_{PGA} and $PGA_{B\text{-}C}$ are obtained as indicated in Section 12.7.2 and Step 1, respectively.
3	The design spectral response acceleration, S_a, for periods, 0.00 < T ≤ 0.20 second is computed using the following equation. $$S_a(T) = F_a S_{\leq 0.20}$$ Equation 12-42 Where $S_{\leq 0.20}$ includes PSA for periods, T = 0.08 sec (13Hz), 0.15 sec (6.7Hz), and 0.20 sec (5Hz) from Step 1. The site factor F_a is obtained as indicated in Section 12.7.3
4	The design spectral response acceleration, S_a, for periods, 1.0 ≤ T ≤ 3.0 second is computed using the following equation. $$S_a = F_v S_{\geq 1.0}$$ Equation 12-43 Where $S_{\geq 1.0}$ includes PSA for 1.0 sec (1.0Hz) and 2.0 sec (0.5Hz). The site factor F_v is obtained as indicated in Section 12.7.3.
5	The spectral accelerations, S_a, for periods, 0.20 < T < 1.0 sec should be linearly interpolated between $S_{0.20}$ at T= 0.20 seconds and $S_{1.0}$ at T = 1.0 second. Where $S_{0.20}$ and $S_{1.0}$ are obtained as indicated in Steps 3 and 4, respectively.

After the Multi-Point horizontal ADRS curve has been constructed, the following should be checked to see if the Three-Point ADRS curve is underestimating spectral accelerations or not representative of the acceleration response spectrum.

- If fundamental periods of vibration greater than 1.0 second are important to the structural response, check Multi-Point spectral acceleration, S_a, corresponding to the 2.0 second period to assure that the long-period response is not underestimated.

- If fundamental periods of vibration less than 0.20 seconds are important to the structural response, check Multi-Point spectral acceleration, S_a, corresponding 0.10 sec period to assure that the short-period response is not underestimated.

- Check to see if the general trend of the Three-Point ADRS curve is similar to the Multi-Point ADRS curve. In certain circumstances there may be a shift that is not captured by the Three-Point ADRS, this is particularly true in the Eastern United States where the peak of the acceleration response spectrum is shifted towards the

1.0 second period. This shift appears to occur at project sites where the soil column is significantly deep and the site stiffness is $\overline{V}_S$ < 600 ft/sec. If the fundamental period of the structure is in the range of longer periods the spectral accelerations will be significantly underestimated using the Three-Point ADRS.

If discrepancies between the Three-Point method and the Multi-Point method have the potential to significantly underestimate the spectral response, the PCS/GDS must be contacted. The PCS/GDS will either approve modifications to the Three-Point ADRS curve or require a site-specific response analysis.

The ADRS curves in Figure 12-36 provide an example where discrepancies between the Three-Point method and the Multi-Point method indicate spectral accelerations (Sa) significantly underestimated at the 1.0 second period and significantly dissimilar acceleration response spectrum shape. The bridge location had a Site Class E and the fundamental period of the structure was 1.0 second. A site-specific response analysis was performed in accordance with Section 12.8 and the Site-Specific ADRS curve was generated for this example as shown in Figure 12-39.

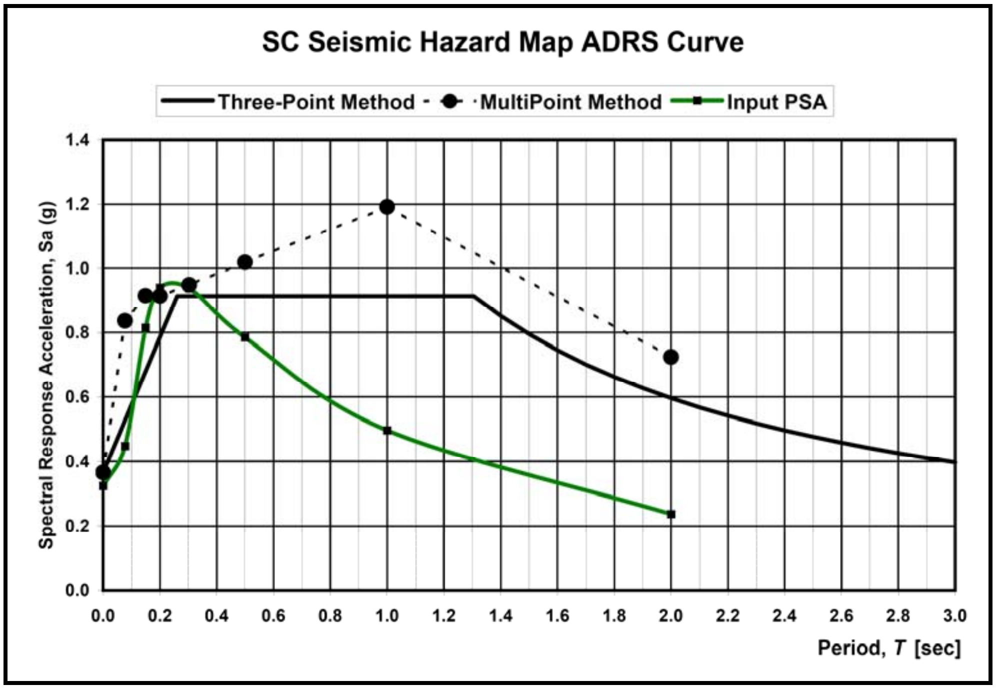


Figure 12-36, Three-Point and Multi-Point Method Comparison (Site Class=E)

12.7.6 <u>ADRS Evaluation using SC Seismic Hazard Maps</u>

Even though ADRS determination using SC Seismic Hazard maps is relatively straight forward, a series of checks are necessary to ensure its appropriateness. This involves using the Three-Point method as the basis of developing the ADRS curve and the Multi-Point method to confirm its validity. A decision flow chart is shown in Figure 12-37 to assist the designer with developing the ADRS curve based on SC Seismic Hazard map.

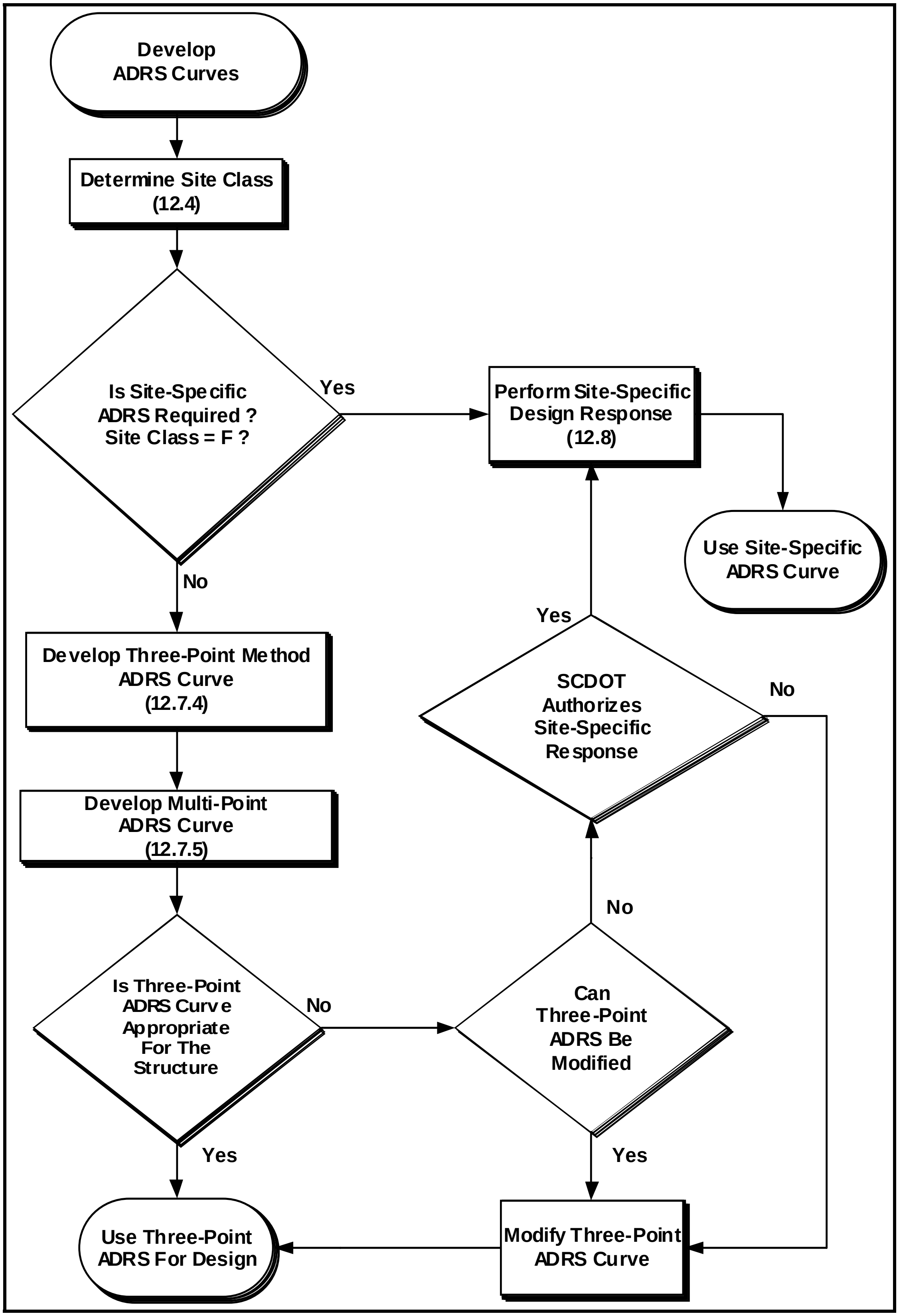


Figure 12-37, ADRS Curve Development Decision Chart

12.7.7 Damping Modifications of Horizontal ADRS Curves

The horizontal acceleration design response spectrum (ADRS) curves developed using the SC Seismic Hazard maps are based on a damping ratio of 5 percent. ADRS curves for structural damping ratios other than 5 percent can be obtained by multiplying the 5 percent damped ADRS curve by the period-dependent factors shown in Table 12-31. For spectra constructed using the Three-Point method, the factors for periods of 0.20 sec and 1.0 sec can be used.

Table 12-31, Damping Adjustment Factors
(Newmark and Hall, 1982, Abrahamson, 1993, and Idriss, 1993)

Period (seconds)	Ratio of Response Spectral Acceleration for Damping Ratio ξ to Response Spectral Acceleration for ξ_{eff} = 5%		
	ξ_{eff} = 2%	ξ_{eff} = 7%	ξ_{eff} = 10%
0.02	1.00	1.00	1.00
0.10	1.26	0.91	0.82
0.20	1.32	0.89	0.78
0.30	1.32	0.89	0.78
0.50	1.32	0.89	0.78
0.70	1.30	0.90	0.79
1.00	1.27	0.90	0.80
2.00	1.23	0.91	0.82
4.00	1.18	0.93	0.86

12.8 SITE-SPECIFIC RESPONSE ANALYSIS

The site-specific response analyses requirements in this section apply only to “Typical SCDOT Bridges” as defined by Sections 1.4 and 1.5 of the SCDOT *Seismic Design Specifications for Highway Bridges*. For non-typical bridges, the PCS/GDS will specify and/or approve appropriate geotechnical earthquake engineering provisions on a project specific basis. The site-specific response analysis is required when any of the following conditions are met.

- Structure has a Site Class F (Section 12.4)
- SC Seismic Hazard Maps are not appropriate (Section 12.7.5 and 12.7.6)
- As required by SCDOT

Site-specific ADRS curves that are generated using a non-linear effective stress site response software such as indicated in Sections 12.8.2 shall model the soils in both a liquefied and non-liquefied configuration and develop an ADRS envelope that combines the maximum spectral response amplifications for the site.

12.8.1 Equivalent-Linear One-Dimensional Site-Specific Response

An equivalent–linear one-dimensional site-specific response analysis shall be performed using SHAKE91 or other computer software that is based on the SHAKE91 computational model. The SHAKE91 computer program models a soil column with horizontal layered soil deposits overlying a uniform visco-elastic half space. The SHAKE91 computer program is based on the original SHAKE program developed by Schnabel, et al. (1972) and updated by Idriss and Sun

(1992). The computer program DeepSoil (Hashash et al., 2005) has been developed specifically for the CEUS and performs the equivalent linear analysis similar to Shake91. Requests to use software other than SHAKE91 or DeepSoil to perform the site-specific response analysis shall be made in writing to the PCS/GDS. Approval to use an alternate site-specific response analysis program shall be dependent on the software being nationally recognized in the United States as SHAKE91 type software and the designer is able to demonstrate project-specific experience using the proposed software.

For most projects and site conditions, the SHAKE91 method (or equivalent) of performing a site-specific response analysis will be required. When this method cannot accurately capture or model the site response, a non-linear one-dimensional effective stress site-specific response analysis may be required by the PCS/GDS. Situations where an equivalent–linear one-dimensional site-specific response analysis (SHAKE91) method has been shown to be unreliable are listed below:

- When ground-shaking levels are greater than 0.4g or if calculated peak shear strains exceed approximately 2 percent.
- When sites have significant liquefaction potential.
- When the non-linear mass participation factor (r_d) indicates either very low site stiffness, $V^*_{S,40'} <$ 400 ft/sec (120 m/sec) or very high site stiffness, $V^*_{S,40'} >$ 820 ft/sec (250 m/sec) and the project site has soil layers that have been screened to be potentially liquefiable.
- When seismic slope instability evaluations are required where complex geometries exist such as compound slopes, broken back slopes, or excessively high earth structures (embankments, dams, earth retaining systems).
- When sites have sensitive soils ($S_t > 8$).

12.8.2 One-Dimensional Non-Linear Site-Specific Response

The PCS/GDS must authorize the use of a non–linear one-dimensional effective stress site-specific response analysis. Guidance in using non-linear site response analysis procedures can be obtained from Kwok et al. (2007). One-dimensional non-linear site response analyses shall be performed using approved computer software such as DESRA-2 (Lee and Finn, 1978) that models the behavior of the soil subjected to cyclic loadings by tracing the evolution of the hysteresis loops generated in a soil by cyclic loading in a sequential manner. A number of other software programs such as D-MOD (Matasovick, 1993), DESRA-MUSC (Qiu, 1998), and DeepSoil (Hashash et al., 2005) have been developed that modify and improve the accuracy of the constitutive soil models originally developed. Authorized software used to perform one-dimensional non-linear site-specific response analysis must be based on the original DESRA-2 by Lee and Finn (1978) or equivalent. Requests to use software other than those indicated above to perform the non-linear site-specific response analysis shall be made in writing to the PCS/GDS. Approval to use an alternate non-linear site-specific response analysis program shall be dependent on the software being nationally recognized in the United States and the designer is able to demonstrate project-specific experience using the proposed software.

12.8.3 Earthquake Ground Motion

The SC Probabilistic Seismic Hazard study computer program Scenario_PC (2006) will be used to generate synthetic ground motions. The time histories generated by Scenario_PC (2006) are described in Section 11.8.4. The time history generated is a synthetic motion that can be matched to the uniform hazard or scaled to a period or frequency range of structural significance. Since a linear elastic time history dynamic analysis is being performed, a single time history matching the Uniform Hazard Spectrum will generally be sufficient for the majority of projects, particularly those located in the Coastal Plains. As indicated in Section 11.8.4, additional time histories may be needed based on the deaggregation results. Additional time histories may be required by SCDOT if project and site conditions warrant it.

12.8.4 Site Characterization

A one-dimensional soil column model is needed when performing a site-specific response analysis. The soil column extends from the location where the ground motion transmits the ground shaking energy to the structure being designed (depth-to-motion, Z_{DTM}, see Section 12.4.2) to the bedrock or geologically realistic site condition (B-C Boundary), where the ground motion has been developed.

When performing an equivalent–linear one-dimensional site-specific response analysis, the soil layers in the one-dimensional column are characterized by the Total Unit Weight (γ_{TW}), Shear Wave Velocity (V_s), Shear Modulus Reduction Curves (Normalized Shear Modulus, G/G_{max} vs. Shear Strain, γ), and Equivalent Viscous Damping Ratio Curves (Soil Damping Ratio, D vs. Shear Strain, γ). These soil parameters are described in Section 12.3. The soil column model should be prepared in tabular form similar to Table 12-32.

Table 12-32, One-Dimensional Soil Column Model

Geologic Time	Layer No.	Layer Thickness, H	Soil Formation	Soil Description (USCS)	PI	FC	Total Unit Weight, γ_{TW}	Shear Wave Velocity, V_s	Shear[(1)] Modulus Reduction Curve	Equivalent [(1)] Viscous Damping Ratio Curve
Quaternary	1									
	2									
Tertiary	3									
	4									
Cretaceous	5									
	6									
Bed Rock	i									

Note: PI = Plasticity Index; FC=% Passing the #200 sieve
(1) Indicate the cyclic stress-strain behavior method used by indicating reference (i.e. Andrus et al. (2003).

The development of the one-dimensional soil column for a project site may require making several assumptions as to the selection of layer thicknesses and soil properties. Therefore, the geotechnical engineer will need to perform a sensitivity analysis on the one-dimensional soil column model being developed to evaluate the consequences of the following:

- Variation in depth to B-C boundary and/or depth to basement rock
- Variations in soil properties for soils encountered below the maximum depth of the geotechnical investigation.

- Variations in soil properties of soils encountered during the geotechnical investigation across the project site.

The sensitivity analysis methodology must be well developed and documented in detail in the report. As a result of the sensitivity analysis performed, a series of site-specific horizontal acceleration response spectra (ARS) curves may be developed. A single recommended site-specific horizontal ARS curve should be superimposed on the graph. The method of selecting the recommended site-specific ARS curve should be documented in the report. The selection of the recommended site-specific ARS curve may be based on the sum of the squares (SRSS), the arithmetic mean, critical boundary method, or other method deemed appropriate. The method selected to develop the recommended site-specific ARS shall be indicated in the Site-Specific Response Analysis Study. The sensitivity analysis will be required for each time history developed for the project site.

When performing a non-linear one-dimensional effective stress site-specific response analysis the soil column model input motions shall be documented to at least the same level of detail as used in the equivalent-linear one-dimensional site-specific response analysis.

In addition to the site-specific design response report, all electronic input and output files shall be submitted to the PCS/GDS.

12.8.5 Site-Specific Horizontal ADRS Curve

The development of the recommended site-specific horizontal acceleration design response spectra (ADRS) shall be based on results of the site-specific response analysis (Sections 12.8.1 or 12.8.2). The Site-Specific ADRS curve should be developed for an equivalent viscous damping ratio of 5 percent. Additional ADRS curves may be required for other damping ratios appropriate to the indicated structural behavior. When the 5 percent damped Site-Specific ADRS curve has spectral accelerations in the period range of greatest significance to the structural response that are less than 70 percent of the spectral accelerations computed using the Three-Point Method, the PCS/GDS shall be consulted to determine if the spectral accelerations less than the 70 percent criteria can be used or if an independent third-party review of the ADRS curve by an individual with the expertise in the evaluation of ground motions is to be undertaken.

A smoothed Acceleration Design Response Spectrum (ADRS) curve shall be superimposed over the recommended site-specific acceleration response spectrum generated from site-specific response analysis (Sections 12.8.1 or 12.8.2). The steps to develop the smoothed ADRS curve shall be based on Table 12-33 and Figure 12-38.

Table 12-33, Site-Specific ADRS Construction Procedures

Step	Procedure Description
1	The maximum design spectral response acceleration, S_{DMax}, shall be taken as the spectral acceleration from the recommended site-specific acceleration response spectra at a period of 0.20 sec, except that it should not be taken as less than 90 percent of the peak response acceleration at any period.
2	With the plateau established as the value of S_{DMax} obtained from Step 1, graphically select value period markers, T_o and T_s, so as to create a best-fit of the site-specific response curve.
3	For spectral accelerations beyond the period of T_s, a smoothed curve based on Equation 12-44 shall be fitted over the site-specific acceleration response spectrum so that a best-fit is made with the site-specific response data so as not to allow any value to be less than 90 percent of the values obtained using the site-specific acceleration response spectrum. If the limitation of the 70 percent criteria of the Three-Point method is used as the lowest spectral acceleration permitted, the best-fit curve shall be adjusted to include the 70 percent criteria limitation. $$S_a = \frac{n}{e^T}$$ Equation 12-44 Where T is the period in seconds and n is a non-dimensional curve fitting number that is adjusted as required.
4	For periods, T, less than or equal to T_o, the design spectral response acceleration S_a shall be given by the following equation. $$S_a = PGA + \frac{T(S_{DMax} - PGA)}{T_o}$$ Equation 12-45 Where PGA is the spectral acceleration at a period, T = 0 seconds, S_{Dmax} is obtained from Step 1, and T_o is obtained from Step 2.
5	The site-specific response reports shall included the following items: • Recommended site-specific response curve • Smoothed Site-Specific ADRS curve • Table of smoothed ADRS data values (T and S_a) • Table with design spectral response parameters PGA, S_{Dmax}, S_{DS}, S_{D1}, and period markers T_o and T_s, as determined from the smoothed ADRS curve. • Equations 12-44 and 12-45 with all variables documented. An example of the information required is shown in Figure 12-39. The 2-per moving average ARS curve is used as an example, the recommended site-specific ARS curve should be constructed as indicated in Section 12.8.4.

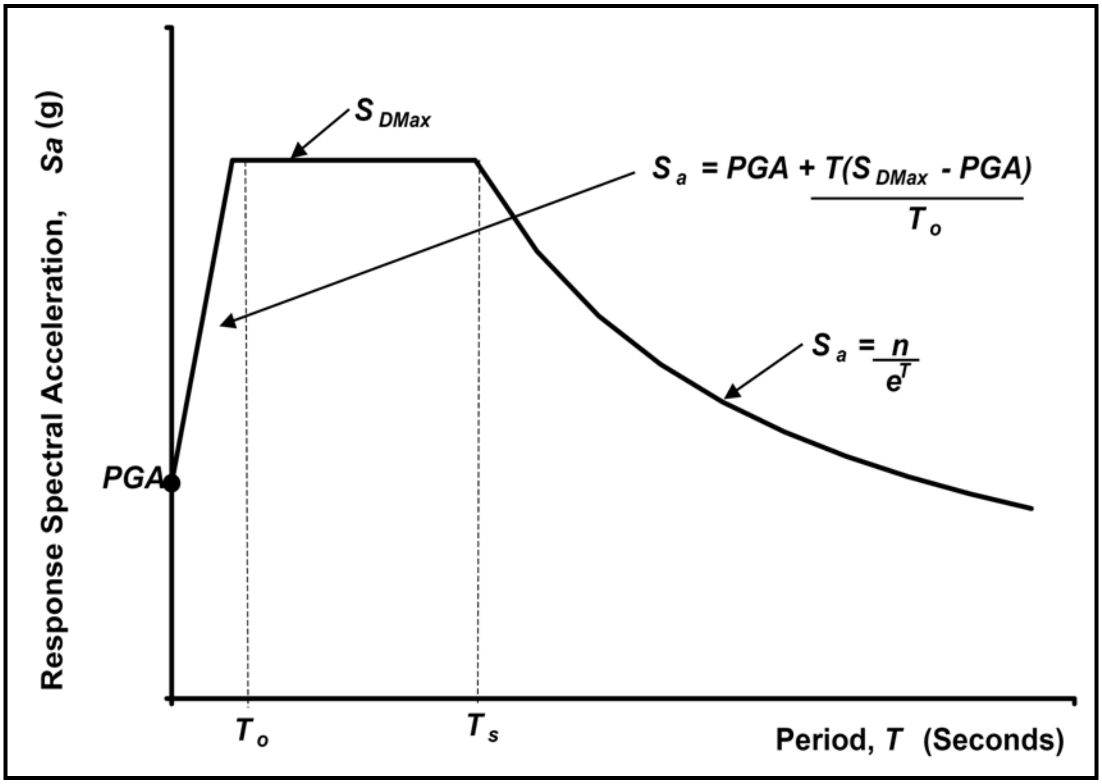


Figure 12-38, Site-Specific Horizontal ADRS Curve Construction

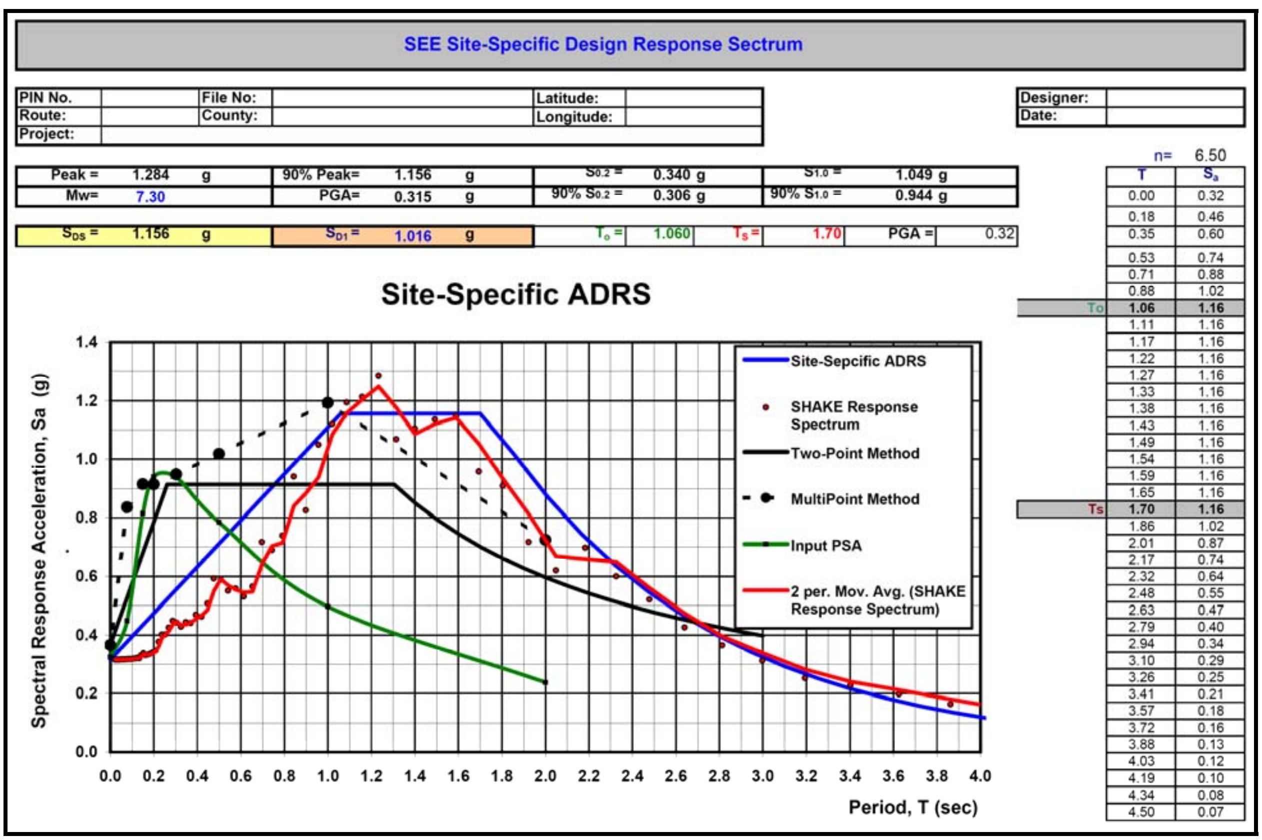


Figure 12-39, Site-Specific Horizontal ADRS Curve (Site Class E)

12.9 GROUND MOTION DESIGN PARAMETERS

12.9.1 Peak Horizontal Ground Acceleration

The peak horizontal ground acceleration (*PGA* or *PHGA*) at the ground surface is defined as the acceleration in the response spectrum obtained at a period, T = 0.0 seconds. If the Three-Point ADRS curves are used, the *PGA* obtained from Section 12.7.2 shall be used. If a site-specific response analysis is performed the spectral acceleration at period T = 0.0 second obtained from Site-Specific ADRS curve should be used.

12.9.2 Earthquake Magnitude / Site-to-Source Distance

The earthquake moment magnitude, M_W, and the site-to-source distance, R, can be obtained from the seismic hazard deaggregations charts discussed in Section 11.8.3.

12.9.3 Earthquake Duration

The earthquake duration is important when evaluating geotechnical seismic hazards that are influenced by degradation under cyclic loading. The longer the duration of the earthquake the more damage tends to occur. Geotechnical seismic hazards that would be affected by degradation under cyclic loading would be sites with cyclic liquefaction potential and liquefaction induced hazards such as lateral spreading and seismic instability.

The SCEC (Southern California Earthquake Center) DMG Special Publication 117 recommends using the Abrahamson and Silva (1996) relationship for rock. The Abrahamson and Silva (1996) correlation between moment magnitude (M_W), site-to-source distance (R), and the earthquake significant duration as a function of acceleration (D_{a5-95}) can be computed by the following equation.

R < 10 km:

Equation 12-46

$$\ln\left(D_{a5-95}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(5.204 + 0.851\left(M_W - 6\right)\right)}{10^{\left(1.5 M_W + 16.05\right)}}\right)^{\left(-\frac{1}{3}\right)}}{15.7x10^{6}}\right] + 0.8664$$

R ≥ 10 km:

Equation 12-47

$$\ln\left(D_{a5-95}\right) = \ln\left[\frac{\left(\dfrac{\exp\left(5.204 + 0.851\left(M_W - 6\right)\right)}{10^{\left(1.5 M_W + 16.05\right)}}\right)^{\left(-\frac{1}{3}\right)}}{15.7x10^{6}} + 0.063\left(R - 10\right)\right] + 0.8664$$

Where:

M_W	=	Moment magnitude of design earthquake (FEE or SEE) Section 12.9.2
R	=	Site-to-source distance (kilometers) Section 12.9.2

Kempton and Stewart (2006) developed a ground motion prediction equation to estimate the earthquake significant duration as a function of acceleration ($D_{a5\text{-}95}$) by using a modern database and a random-effects regression procedure. The correlation presented in the following equation uses the earthquake moment magnitude (M_W), site-to-source distance (R), site stiffness ($\overline{V}_S = V_{S,30}$), and depth-to-hard rock (Z_{HR}) to estimate the earthquake significant duration ($D_{a5\text{-}95}$).

Equation 12-48

$$\ln\left(\boldsymbol{D_{a5-95}}\right) = \ln\left[\frac{\left(\frac{\exp\left(\mathbf{2.79 + 0.82}\left(\boldsymbol{M_W} - \mathbf{6}\right)\right)}{\mathbf{10}^{\left(\mathbf{1.5}\boldsymbol{M_W} + \mathbf{16.05}\right)}}\right)^{\left(-\frac{1}{3}\right)}}{\mathbf{15.68 \cdot 10^6}} + \mathbf{0.15}\boldsymbol{R} + \mathbf{2.53} - \mathbf{0.0041}\boldsymbol{\overline{V}_S} + \mathbf{1.2 \cdot 10^{-3}}\boldsymbol{Z_{HR}}\right] + \varepsilon$$

Where:

$\overline{V}_S$	=	Site stiffness with Z_{DTM}=0 (Section 12.3.4)
M_W	=	Moment magnitude of design earthquake (FEE or SEE) Section 12.9.2
R	=	Site-to-source distance (kilometers) Section 12.9.2
Z_{HR}	=	Depth from ground surface to hard rock (Vs >5,000 ft/sec (1,500 m/s)) Units of
ε	=	Near-fault forward directivity correction for earthquakes (dip-slip or strike-slip faults)

R < 20 km: $\varepsilon = \mathbf{0.015}\left(\boldsymbol{R} - \mathbf{20}\right)$

R ≥ 20 km: $\varepsilon = \mathbf{0}$

The Kempton and Stewart (2006) study confirmed the previous correlations (i.e. Abrahamson and Silva (1996)) that earthquake duration (D) increased with an increase in moment magnitude (M_W) and site-to-source distance (*R*). In addition, the study found that the earthquake duration (D) significantly increased with decreasing site stiffness ($\overline{V}_S = V_{S,30}$). The earthquake duration (D) also increased slightly with an increase of depth-to-hard rock (Z_{HR}).

South Carolina shear wave profiles have indicate that site stiffness ($\overline{V}_S = V_{S,30}$) can vary significantly across the state from a Site Class A(> 5,000 ft/s = 1,500 m/s) to a Site Class E (< 600 ft/s = 180 m/s). The effects of site stiffness on earthquake duration using Kempton and Stewart (2006) relationship have been plotted on Figure 12-40. An earthquake moment magnitude, M_W = 7.3 and a depth-to-hard rock, Z_{HR} = 2,600 feet (800m) have been selected as typical of the lower South Carolina Coastal Plain. The Abrahamson and Silva relationship for rock has also been plotted for reference.

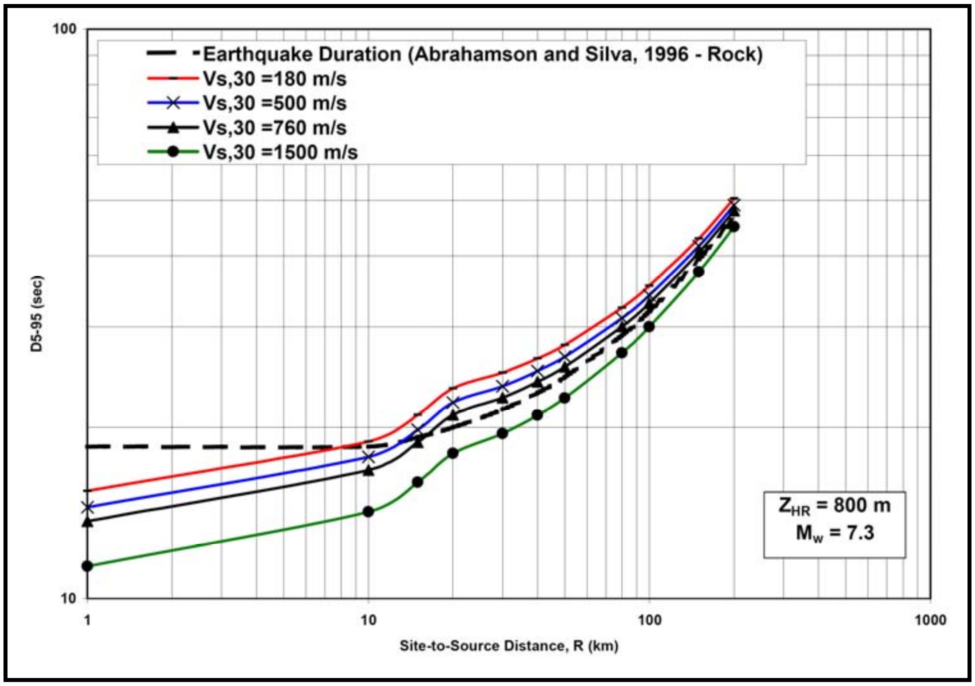


Figure 12-40, Effects of Site Stiffness on Earthquake Duration

South Carolina Coastal Plain geology (Chapter 11) indicates that the depth-to-hard rock varies from zero at the "Fall-line" up-to 4,000 feet (1,200 meters) at the southeastern corner of the state. The effects of depth-to-hard rock on earthquake duration using Kempton and Stewart (2006) relationship have been plotted on Figure 12-41. The Abrahamson and Silva relationship for rock has also been plotted as a reference.

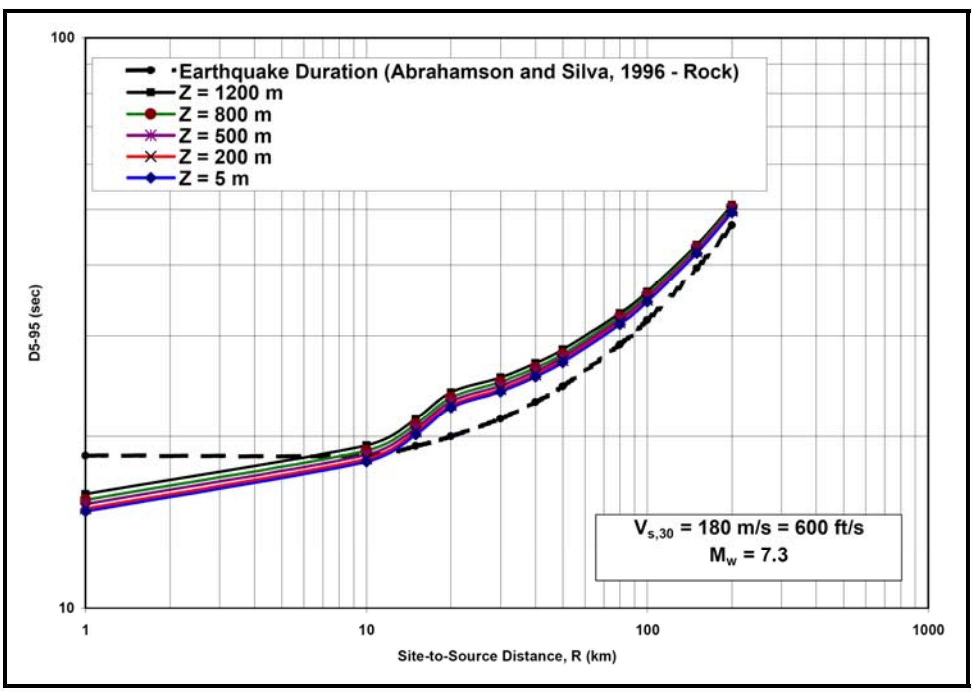


Figure 12-41, Effects of Depth-to-Hard Rock on Earthquake Duration

The project site conditions should be evaluated and the most appropriate earthquake duration model should be used.

12.9.4 Peak Ground Velocity

The peak ground velocity, V_{Peak}, of the earthquake can be determined from a site-specific response analysis. If the Three-Point ADRS curves are developed, peak ground velocity, V_{Peak}, correlations based on the NCHRP 12-70 document may be used.

The peak ground velocity, V_{Peak}, in units of in/sec can be computed by the following equation.

$$V_{Peak} = PGV = 55 F_v S_1 \qquad \text{Equation 12-46}$$

Where,

F_v = site coefficient defined in Table 12-28, based on the Site Class and the mapped spectral acceleration for the long-period, S_1.

S_1 = the mapped spectral acceleration for the one second period as determined in Sections 12.5 and 11.8.2 at the B-C boundary

12.10 REFERENCES

The geotechnical design specifications contained in this Manual must be used in conjunction with the *AASHTO* LRFD Bridge Design Specifications (latest edition). The SCDOT Seismic Design Specifications for Highway Bridges will take precedence over AASHTO seismic guidelines.

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