

Chapter 12

**GEOTECHNICAL
SEISMIC ANALYSIS**

GEOTECHNICAL DESIGN MANUAL

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CHAPTER 12

GEOTECHNICAL SEISMIC ANALYSIS

12.1 INTRODUCTION

Geotechnical seismic analysis consists of evaluating the seismic hazard and the effects of the hazard on the transportation structure being designed. This is accomplished by characterizing the subsurface soils, determining the seismic 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 the potential for collapse during a seismic event. The latest edition of the SCDOT Seismic Specs establishes the seismic design requirements for the design of bridges on the South Carolina highway transportation system. This Chapter presents geotechnical seismic analysis requirements for evaluating ground shaking using either the Seismic Hazard Mapping study or by performing a Site-Specific Response Analysis (SSRA). Determining the potential for soil strength losses, analyzing the hazard caused by reduced soil strengths, and analyzing seismic lateral loadings are contained in Chapters 13 and 14.

The OES/GDS performs the following types of geotechnical seismic engineering analyses:

1. Determine Seismic Design Parameters – PGA, PSA, M_w , R, etc. (Chapter 11)
2. Develop Acceleration Design Response Spectrum (ADRS) curves (Chapter 12)
3. Generate Seismic Ground Motions - Time Histories (Chapter 11)
4. Review Consultant Geotechnical Seismic Engineering Reports (Chapter 21)

Based on the information obtained from the above analyses, the GEOR performs the following geotechnical seismic engineering analyses:

1. Perform Seismic Hazard Analyses – SSL, etc. (Chapter 13)
2. Perform Geotechnical Seismic Engineering Design (Chapter 14)

12.2 GEOTECHNICAL SEISMIC ANALYSIS

The geotechnical analysis requirements for determining the seismic hazard and associated site response have been developed for the design of “typical” bridges as defined by the Seismic Specs. Bridges not meeting the definition of “Typical SCDOT Bridges” include suspension bridges, cable-stayed bridges, arch type bridges, movable bridges, and bridges with spans exceeding 300 feet. For these “non-typical” bridges, the OES/GDS in conjunction with the OES/SDS will specify and/or approve appropriate geotechnical seismic engineering provisions on a project specific basis. The geotechnical seismic analysis requirements in this Manual shall also apply to the design of bridge embankments, ERSs, and other miscellaneous transportation related structures. The Seismic Specs limit the applicability of the 2-level (i.e., designing using both FEE and SEE) design to select bridges that meet specific criteria contained in the Seismic Specs. All bridge embankments (unreinforced, reinforced and RSS) and ERSs located within bridge embankments are required to be designed using both events. ERSs located within roadway

embankments shall only be designed for the SEE. As indicated previously, roadway embankments (unreinforced, reinforced and RSS) will not be designed for the EE I limit state.

The preliminary geotechnical engineering report (PGER) typically contains a geotechnical seismic hazard analysis that includes the ADRS curve to be used for preliminary design of the bridge structure. The final bridge or roadway geotechnical engineering report (BGER or RGER) contains the results of the final geotechnical subsurface investigation and modifies, if necessary, the ADRS curves.

12.3 DYNAMIC SITE PROPERTIES

12.3.1 Soil Properties

A project specific subsurface geotechnical investigation shall be performed 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. These 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, specifically compression and shear wave velocities, V_p and V_s , shall be measured in the field (Chapter 5). Correlation as indicated in Chapter 7 may only be used when insufficient field measurements are available for the development of the site factors as indicated in this Chapter. Other dynamic properties such as shear modulus curves, damping ratio curves, and the residual strength of soils that lose shear strength during the seismic event are determined as indicated in Chapter 7.

12.3.2 Site Stiffness

Site stiffness ($V_{s,H}^*$), as used in this Manual, is a weighted average of the measured soil stiffness of individual soil layers to a specific depth of interest (H). The measured V_s values shall not be corrected for overburden pressure. The weighted average shall be computed using the measured V_s obtained during the geotechnical site investigation. As an alternate, when V_s has not been obtained, V_s may be correlated using SPT resistances or CPT values as indicated in Chapter 7; however, written approval of the OES/GDS shall be obtained prior to using the correlations in Chapter 7. The SPT or CPT correlated V_s values will be determined as required for use in Chapter 13.

Site stiffness shall be computed from measured shear wave velocities as indicated in the following equation.

$$V_{s,H}^* = \frac{H}{t_d} \quad \text{Equation 12-1}$$

Where,

- $V_{s,H}^*$ = Weighted, average site stiffness to a specific depth of interests, typically either the B-C Boundary, Weathered Rock, or Hard Rock basement outcrop, ft/sec
- H = Total depth where V_s is being averaged, typically either the B-C Boundary, Weathered Rock, or Hard Rock basement outcrop, feet
- t_d = Time that it takes for the shear wave to travel from the H to the ground surface, seconds

For layered profile, $V_{s,H}^*$ may also be computed by

$$V_{s,H}^* = \frac{H}{\sum_{i=1}^n \left(\frac{H_i}{V_{si}} \right)} \quad \text{Equation 12-2}$$

Where,

$V_{s,H}^*$ = Weighted, average site stiffness to a specific depth of interest, H, ft/sec

H = Total depth where V_s are being averaged, feet

V_{si} = Shear wave velocity of layer i, ft/sec

H_i = Thickness of any layer i between the ground surface, 0, and H, feet

See Chapter 11 for definitions of B-C Boundary, Weathered Rock (i.e., Geologically Realistic) and Hard Rock. Appendix H provides V_s profiles for various locations in South Carolina. These profiles are included for reference only. Site specific V_s profiles shall be used for the upper 100 feet (30 meters) of a site profile. Deeper, beyond 100 feet, V_s profiles are available for select areas of South Carolina, see the Geotechnical Design Webpage of the SCDOT Website, for select locations where deeper V_s profiles are available. The GIS map available includes links to a PDF of the boring log and an Excel spreadsheet with the available V_s and/or V_p data.

12.3.3 Equivalent Uniform Soil Profile Period and Stiffness

The thickness of the soil deposit, H, above the B-C Boundary, Weathered Rock or Hard Rock and average site stiffness, $V_{s,H}^*$, are used to compute the natural period of the site, T_N , as indicated below. H typically begins at the ground surface, but may begin at the depth where the ground motion is of interest to the structure being designed (see Section 12.5), and extends to the depth where the motion is being generated, typically either the B-C Boundary, Weathered Rock or a Hard Rock basement outcrop (see Chapter 11). The B-C Boundary is the depth below which the V_s remains consistently either equal to or greater than 2,500 feet per second. The depth to top of Weathered Rock is the depth at which V_s remains consistently equal to or greater than 8,200 feet per second, but less than 11,500 feet per second. As indicated previously, the B-C Boundary and Weathered Rock represent Geologically Realistic site conditions in the Coastal Plain or Piedmont Physiographic Provinces, respectively. The depth to top of Hard Rock is the depth at which V_s remains equal to or greater than 11,500 feet per second.

A comprehensive evaluation of how to determine the fundamental period of the soil profile has been made by Dobry, Oweis, and Urzua (1976). Dobry, et al. (1976) presented 2 methods for determining T_N . The first is a simplified procedure, typically used for uniform soil conditions, as presented in Equation 12-3. The second is a more complex method but is still relatively simple and more accurate method to determine the fundamental period of the soil profile and consists of using the Successive 2-Layer Approach proposed by Madera (1970). Hadjian (2002) presented a simplification to the Successive 2-Layer Approach by Madera (1970). It should be noted that the simplified procedure could be as much as 20 percent greater than the Successive 2-Layer Approach according to Vijayendra, Parsad, and Nayak (2010). According to Bray and Travarasrou (2007), T_N may degrade as the site softens during the seismic event. During the seismic event T_N may increase by as much as 50 percent when compared to the T_N generated prior to the seismic event. Equation 12-3 indicates the unsoftened natural site period, while Equation 12-4 indicates the softened site period.

$$T_{NB-C} = \frac{4 * H_{B-C}}{V_{s,H_{B-C}}^*} \tag{Equation 12-3}$$

$$T_{NB-C} = \frac{6 * H_{B-C}}{V_{s,H_{B-C}}^*} \tag{Equation 12-4}$$

Where,

T_{NB-C} = Natural site period measured from the B-C Boundary, Weathered Rock, or Hard Rock basement outcrop, second

$V_{s,H}^*$ = Equivalent uniform soil profile stiffness of thickness (H), ft/sec (Section 12.3.2)

H = Thickness of soil deposit above B-C Boundary, Weathered Rock, or Hard Rock basement outcrop depending on the level where ground motion input has been developed, feet

As can be seen by Equations 12-3 and 12-4, the T_N is influenced by the $V_{s,H}^*$ and H. A general trend is observed in Figure 12-1 that T_N decreases as the site stiffness increases while keeping the soil deposit thickness the same. In addition, as H increases (keeping the $V_{s,H}^*$ the same), the T_N of the site increases. Consequently, a combination of lower $V_{s,H}^*$ and increased H will work together to increase the T_N of the site. At the same time, a reduction in the T_N of the site is observed primarily when the $V_{s,H}^*$ increases as H decreases.

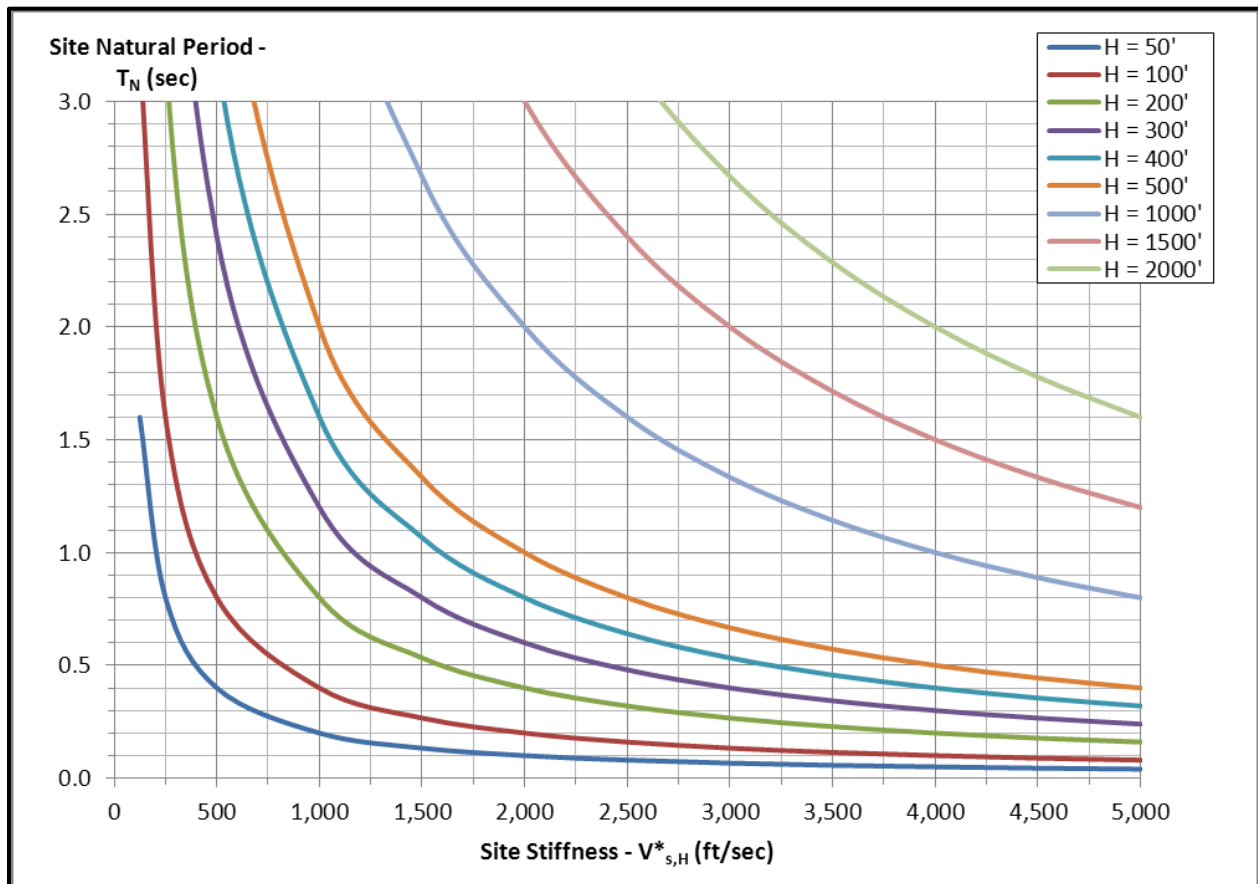


Figure 12-1, Site Stiffness ($V_{s,H}^*$) vs. Site Natural Period (T_N)

The Successive 2-Layer Approach consists of solving for the fundamental period of 2 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 or natural period, T_N . The Successive 2-Layer Approach as modified by Hadjian (2002) to compute the equivalent uniform soil profile period, T_N , and stiffness, $V_{s,H}^*$, is provided in Table 12-1.

**Table 12-1, Modified Successive 2-Layer Approach
(Modified Hadjian (2002))**

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 (H). Compute the periods, T_A and T_B where $A = n$ (i.e., 1) and $B = n+1$ (i.e., 2) using Equations 12-3 and 12-4 in order to provide a range of potential site periods.
2	Beginning at the same point in Step 1 determine the following ratio: $\frac{\gamma_A * H_A}{\gamma_B * H_B} \quad \text{Equation 12-5}$ Where: γ_A = Unit weight of layer 1, pounds per cubic foot γ_B = Unit weight of layer 2, pounds per cubic foot H_A = Thickness of layer 1, feet H_B = Thickness of layer 2, feet
3	Determine the ratio of thickness of consecutive layers: $\frac{H_A}{H_B} \quad \text{Equation 12-6}$ If the ratio is greater than 1 (> 1.0) go to Step 4. If the ratio is less than or equal to 1 (≤ 1.0) go to Step 5.
4	Compute the period for combined layers A and B, T_{A-B} , using the following equation: $T_{A-B} = T_B * \sqrt{\frac{\pi^2}{8} * \left\{ 0.75 + \left(\frac{T_B}{T_A}\right)^2 * \left[1 + 2 * \left(\frac{\gamma_A * H_A}{\gamma_B * H_B}\right) \right] \right\}} \quad \text{Equation 12-7}$
5	Compute the period for combined layers A and B, T_{A-B} , using the following equation: $T_{A-B} = T_B * \sqrt[N]{1.0 + \beta * \left(\frac{T_B}{T_A}\right)^N * \left(1 + \frac{\gamma_A * H_A}{\gamma_B * H_B}\right)^N} \quad \text{Equation 12-8}$ Where, $\beta = 1 - 0.2 * \left(\frac{H_A}{H_B}\right)^2 \quad \text{Equation 12-9}$ $N = 4 - \frac{1.8 * H_A}{H_B} \quad \text{Equation 12-10}$
6	Repeat from Step 2 until the entire soil column has been analyzed, substituting ($\gamma_{A-B} * H_{A-B}$) for $\gamma_A * H_A$, H_{A-B} for H_A , and T_{A-B} for T_A each time.

12.3.4 $V_{s,H}^*$ Variation Along a Project Site

If the $V_{s,H}^*$ varies between the interior bents and abutments of a bridge, the $V_{s,H}^*$ used in the design of the bridge structure must be evaluated jointly between the SEOR and the GEOR. 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 $V_{s,H}^*$ at the bridge abutment will govern. The $V_{s,H}^*$ for longer bridges may differ significantly along the bridge alignment due to variability in soil conditions such as when an abutment is founded on rock ($V_{s,H}^* > 2,500$ ft/sec), the other abutment is founded on soft soils ($V_{s,H}^* < 600$ ft/sec), and the interior bents are founded on stiff soils ($V_{s,H}^* \approx 1,250$ ft/sec). 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 ADRS curves developed using the various $V_{s,H}^*$ values. If the structural analytical method allows the input of several motions at different locations, then several ADRS curves should be used.

The GEOR is responsible for evaluating soil conditions and the extent of site variability (if any) at the bridge location and then determining the $V_{s,H}^*$ for each individual soil region based on the guidelines provided in this Section. The SEOR and the GEOR will then jointly evaluate the appropriate ADRS curve to be used for the structural design.

12.3.5 South Carolina Reference $V_{s,H}^*$

A $V_{s,H}^*$ was computed for the USGS Shear Wave Velocity Data (Odum, Williams, Stephenson, and Worley (2003) and South Carolina Emergency Management Division (URS (2001)) based on the shear wave reference profiles in Appendix H. The reference $V_{s,H}^*$ was determined for each shear wave profile using a $V_{s,H}^*$ computed in accordance with Section 12.3.2 at the ground surface. The $V_{s,H}^*$ for the USGS Shear Wave Velocity Data are provided in Tables 12-2 and 12-3.

**Table 12-2, USGS Site Stiffness
(Modified Odum, et al. (2003))**

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

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

²Longitude is negative indicating west.

The $V_{s,H}^*$ for the SCEMD Seismic Risk and Vulnerability Study are provided in Table 12-3.

**Table 12-3, USGS Site Stiffness
(Modified URS Corporation (2001))**

Site ⁽¹⁾ No.	Site Response Category ⁽¹⁾	Geology	Site Stiffness $V_{s,H}^*$	
			(m/s)	(ft/sec)
1, 2, 4 ⁽²⁾	Piedmont/Blue Ridge, Savannah River, Myrtle Beach ⁽²⁾	Crystalline	3,400	11,152
1	Piedmont/Blue Ridge	Piedmont/Blue Ridge	453	1,486
2	Savannah River	Savannah River	355	1,165
3	Charleston	Charleston	328	1,077
4	Myrtle Beach	Myrtle Beach	239	784

¹ Site Response Categories are shown in Appendix H.

² 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.

12.4 PROJECT SITE CLASSIFICATION

In Versions 1.0 (2008) and 1.1 (2010) of the GDM, the Site Class (A through E) was determined using $V_{s,100}^*$. The Site Class was used to determine the appropriate site amplification factors (F_{PGA} , F_a , or F_v) that were then used to transform the ground motion at the B-C Boundary, Weathered Rock, or Hard Rock basement outcrop to the ground motion at the ground surface. However, according to Andrus, Ravichandran, Aboye, Bhuiyan, and Martin (2014), determining Site Class is no longer required, because the site factors will be based directly on the $V_{s,H}^*$ as measured on the site (see Chapters 4, 5, 6 and Section 12.3). However, the use of the term B-C Boundary will continue even though the Site Classes B and C will no longer be used. B-C Boundary as used in the GDM indicates that the mean (average) $V_{s,H}^*$ is in excess of 2,500 feet/second and is no more than 1 standard deviation (σ) less than this value (-1σ) from the point where $V_{s,H}^* = 2,500$ feet/second is encountered. The B-C Boundary shall be moved to a deeper depth if the shear wave velocity profile is more than -1σ from 2,500 feet per second. The GEOR shall determine the depth to the B-C Boundary based on available data using the Geotechnical Design Webpage previously discussed in Section 12.3.2. In addition, Site Class F (sites requiring Site-Specific Response Analyses) shall continue to be used.

The $V_{s,H}^*$ used in the determination of the site amplification factors for different periods (F_t) shall be computed in accordance with Section 12.3.2. The H where V_s will be analyzed should begin at either the existing ground surface if no fill is present or at the estimated original ground surface beneath the embankment, and extend to a depth of at least 100 feet ($H = 100$ ft.). If the depth-to-motion, Z_{DTM} concept is to be used, the $V_{s,H}^*$ profile shall begin at the Z_{DTM} and extend 100 feet below the Z_{DTM} . The Z_{DTM} is the location where the ground shaking is transmitted to the structure being designed. Guidance in selecting the, Z_{DTM} , is provided in Section 12.5.

When there is a high contrast in V_s in the soil column, the computed $V_{s,H}^*$ may not be representative of the site response. The GEOR will need to evaluate the computed $V_{s,H}^*$ for high variation in V_s within the profile that could potentially overestimate the $V_{s,H}^*$ and in turn miscalculate amplification of the spectral accelerations. The procedure provided in Table 12-4 should be used to evaluate $V_{s,H}^*$ variability and should be used cautiously as only a guide. The GEOR will be responsible for making all $V_{s,H}^*$ recommendations, and these recommendations will be submitted to the OES/GDS for review and acceptance. The proposed procedure to evaluate the $V_{s,H}^*$ variability is based on the potential variability of V_s testing having a COV of 0.10 to 0.20.

Table 12-4, Site Stiffness Variability Proposed Procedure

Step	Description
1	Compute the COV of the V_s values (COV_{V_s}) within the soil profile column. If the COV_{V_s} is greater than 0.10 but less than or equal to 0.30 proceed to Step 2. For COV_{V_s} greater than 0.30 proceed to Step 3. If the $COV_{V_s} \leq 0.10$ then compute the $V_{s,H}^*$ using the V_s values in accordance with Section 12.3.
2	<p>If $0.10 < COV_{V_s} \leq 0.20$ adjust $V_{s,H}^*$ using Equation 12-11 then proceed to Step 3.</p> $V_{s,H,\leq 0.2}^* = V_{s,H}^* * (1 - 0.20) \quad \text{Equation 12-11}$ <p>If $0.20 < COV_{V_s} \leq 0.30$ adjust $V_{s,H}^*$ using Equation 12-12 then proceed to Step 3.</p> $V_{s,H,\leq 0.3}^* = V_{s,H}^* * (1 - COV_{V_s}) \quad \text{Equation 12-12}$
3	If COV_{V_s} is greater than 0.30, the GEOR shall submit to the OES/GDS either a recommended (with documentation) $V_{s,H}^*$ to be used for the project or request a site-specific response analysis be performed in accordance with Section 12.9.

When a project site has variable $V_{s,H}^*$ due to soil spatial variations along the project alignment or when different structural components (bridge abutment, interior bents, embankments, etc.) require differing, Z_{DTM} , the design team will need to evaluate the $V_{s,H}^*$ for each structural component being designed. Guidance in selecting the most appropriate $V_{s,H}^*$ for the structure being designed can be found in Section 12.5.

The following conditions shall be used for determining a Site Class F:

- Peats and/or highly organic clays ($H > 10$ ft [3 m] of peat and/or highly organic clay where H = thickness of soil)
- Very high plasticity clays ($H > 25$ ft [8 m] with $PI > 75$)
- Very thick soft/medium stiff clays ($H > 120$ ft [36 m])
- Soft soil layer ($H > 10$ ft [3 m]); $PI > 20$; $w > 40\%$, and $\overline{s_u} < 500$ psf (25 kPa) {All conditions must be met.}

If the site meets any of these criteria, classify the project site as Site Class F and perform an SSRA. In addition, Kavazanjian, et al. (2012) has further identified sites where the use of the 3-Point method may not be appropriate. These sites include sites with a soil column in excess of 500 feet or where a sharp impedance contrast (i.e., a change in soil stiffness or V_s) occurs within 150 feet of the ground surface. The completed Andrus, et al. (2014) research accounts for both of these additional site conditions. Therefore, a site-specific seismic response analysis will typically not be required for either a soil column with a depth greater than 500 feet or for sites with a sharp impedance contrast within 150 feet of the ground surface (see Section 12.8 for guidance). However, the OES/GDS in consultation with the OES/SDS shall determine whether an SSRA is required.

12.5 DEPTH-TO-MOTION EFFECTS ON SITE CLASS AND SITE FACTORS

For certain types and lengths of bridges it may be more practical to apply the seismic ground motion at a point different from the existing/original ground surface. The types of bridges where changing this depth (depth-to-motion, Z_{DTM}) may be practical are those bridges that are not covered by the Seismic Specs. The length of bridge where changing the Z_{DTM} is beneficial shall be determined by the SEOR with concurrence from the OES/SDS.

An SSRA (Section 12.9) shall be required to determine the ADRS curve, when using Z_{DTM} . It is anticipated that an iterative process will be required between the SEOR and the OES/GDS to determine the Z_{DTM} . In the cases where the Z_{DTM} is used, the OES/GDS shall provide to the SEOR the soil models and the critical penetration (Chapter 16). Once the SEOR has determined a Z_{DTM} , the OES/GDS shall provide the ADRS curve for this depth.

The $V_{s,H}^*$ shall be determined to 100 feet below the Z_{DTM} . This $V_{s,H}^*$ shall be used to determine the 3-point ADRS curve, with this ADRS curve being used for comparison with the ADRS curve from the site-specific seismic response analysis.

12.6 SC SEISMIC HAZARD ANALYSIS

The SC Seismic Hazard study shall be used for all “typical” bridges as defined in the Seismic Specs, as well as, bridge embankments and roadway structures. For “non-typical” bridges, the OES/GDS will specify and/or approve appropriate geotechnical seismic analysis provisions on a project specific basis. The Seismic Hazard Mapping study is described in Chapter 11. 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. The GEC shall obtain a Seismic Information Request form (GDF 002, see Appendix A) and submit it to the OES/GDS. The most current version of this request form is available on the SCDOT website.

The request form (GDF 002) requires that the GEC provide the following information:

- SCDOT Project ID
- County
- RPG
- Route
- Description of Project
- Project latitude and longitude
- Indicate which of the following is also being supplied
 - V_s Profile to B-C Boundary (Geologically Realistic – Coastal Plain)
 - V_s Profile to $V_s \geq 8,200$ feet per second (Geologically Realistic – Piedmont)
 - V_s Profile to $V_s \geq 11,500$ feet per second (Hard Rock)

The GEC is required to provide the V_s profiles indicated above in an Excel® format. The provided V_s profiles shall include the following information in order as presented and shall extend to the B-C Boundary or the top of Weathered Rock as determined by the GEOR. In addition, the GEOR shall note in the V_s profile which deep hole location was used to determine the B-C Boundary or the top of Weathered Rock.

- Depth, feet

- V_s , feet per second
- V_p , feet per second (if measured)
- Density or unit weight, pounds per cubic foot

The GEC, using the “Site Condition” models contained in Chapter 11, is required to provide documentation for the selection of the Site Condition (Geologically Realistic or Hard-Rock basement outcrop) used. Typically, most sites will be Geologically Realistic unless the $V_{s,H}^*$ is over 11,000 ft/sec within the 100-foot soil column. Then the “Site Condition” would be considered to be Hard-Rock.

Upon receipt of a completed Seismic Information Request form from the GEC, the OES/GDS shall use the information to develop a 3-Point ADRS curve in accordance with the requirements of this Chapter.

12.7 ACCELERATION RESPONSE SPECTRUM

The acceleration response spectrum of a specific seismic 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 Seismic Hazard Mapping study generates a probabilistic UHS consisting of the PGA and PSA at either a Hard-Rock basement outcrop or at Geologically Realistic site conditions (i.e., B-C Boundary or Weathered Rock). 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 maximum local site amplification occurs when the predominant or maximum period, T'_o (see Section 12.10.3), of the rock outcrop ground motion, the soil deposit's natural period, T_N , and the fundamental period of the structure, T_0 , 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 percent damped) are shown in Figure 12-2 (Seed, Ugas, and Lysmer (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 (≈ 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 $V_{s,H}^*$ in accordance with Section 12.3.

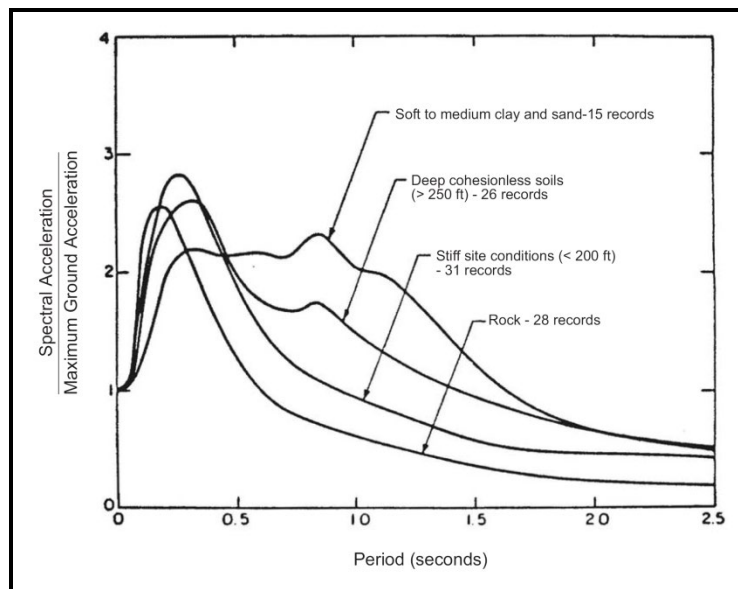
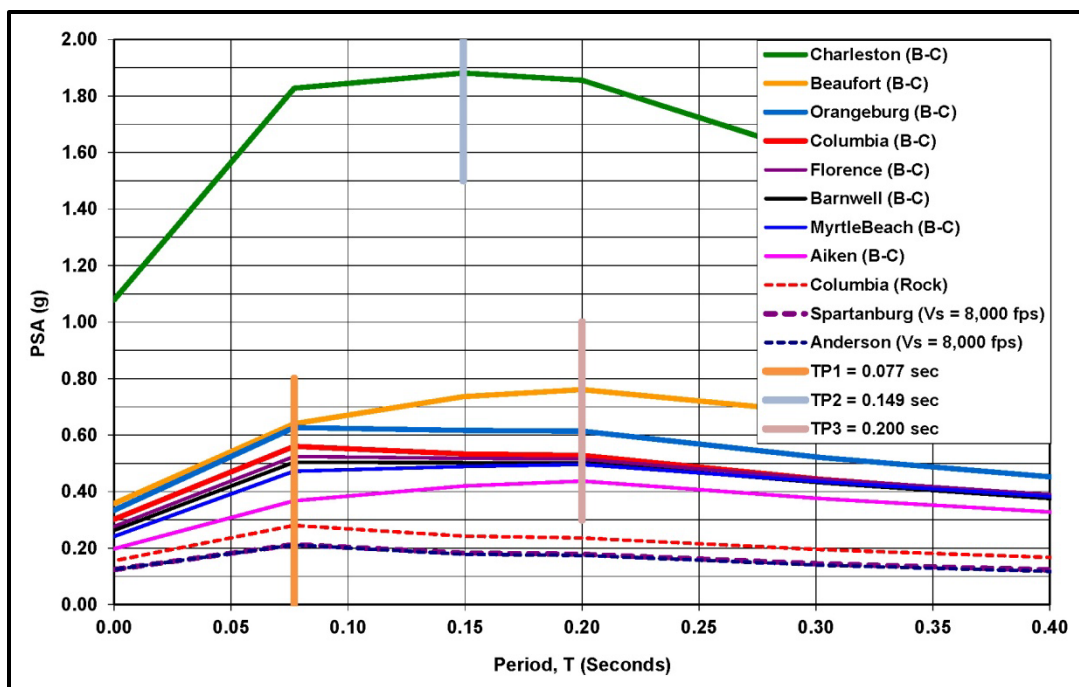


Figure 12-2, Soil Site Effects on Average Normalized Response Spectra (Seed, et al. (1976))

It is equally important to know the fundamental period (first order mode) of the structure (T_0) (i.e., bridge, ERS, 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. T_0 is determined by the SEOR.

A 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. The predominant period (T'_0) for the SEE seismic motion for select South Carolina cities may be obtained from the UHS, see Figure 12-3. The UHS is determined using the Geologically Realistic model, B-C Boundary in the Coastal Plain or Weathered Rock for sites outside of the Coastal Plain or at the Hard-Rock basement outcrop (see Chapter 11 for selection of the appropriate geologic conditions). The difference between Green (2001) and Figure 12-3 is T_{max} was determined for Hard-Rock conditions and did not account for the thickness of the soil deposit on top of the rock.



T'_o = Predominant Period based on the SEE ground motion
 T'_o -1 (TP1) – Anderson, Barnwell, Columbia, Florence, Orangeburg, Spartanburg
 T'_o -2 (TP2) – Charleston
 T'_o -3 (TP3) – Aiken, Beaufort, Myrtle Beach

Figure 12-3, Predominant Period (T'_o) of Selected SC Cities

T'_o , T_N and T_0 should be compared by the SEOR and if these periods coincide then harmonic resonance between the seismic event, the site and the structure should be anticipated. If T'_o and T_N coincide then site amplification should be anticipated. The OES/GDS shall determine if an SSRA is required if T'_o and T_N coincide.

The local site effects are taken into account by performing a site response analysis using the Seismic Hazard Mapping Study (Section 12.8) or by performing an SSRA (Section 12.9). The following Subsections describe special site conditions that may influence the site response that typically cannot be addressed by simplified response methods that use the Seismic Hazard Mapping Study (Section 12.8).

12.7.1 Effects of Rock Stiffness WNA vs. ENA

The effects of rock stiffness (shear wave velocity) and damping on normalized response spectra shapes (5 percent damped) on rock sites are shown in Figure 12-4 (Silva and Darragh (1995)). Normalized spectral shapes were computed by dividing the spectral acceleration by the PGA at the surface. Normalized response spectra were computed for Western North America (WNA), representative of soft rock (i.e., $V_s \cong 5,000$ feet per second) encountered in California and for Eastern North America (ENA), representative of hard rock (i.e., $V_s \cong 11,500$ feet per second) encountered in the Eastern United States. The normalized response spectra were computed from motion records made on rock sites at close distances to earthquakes ($M_w = 4.5$ and 6.4). These normalized spectral curves show that ENA spectral response amplification is greater at shorter periods or higher frequencies when compared to WNA spectral response. This effect of higher amplification at shorter periods or higher frequencies 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)).

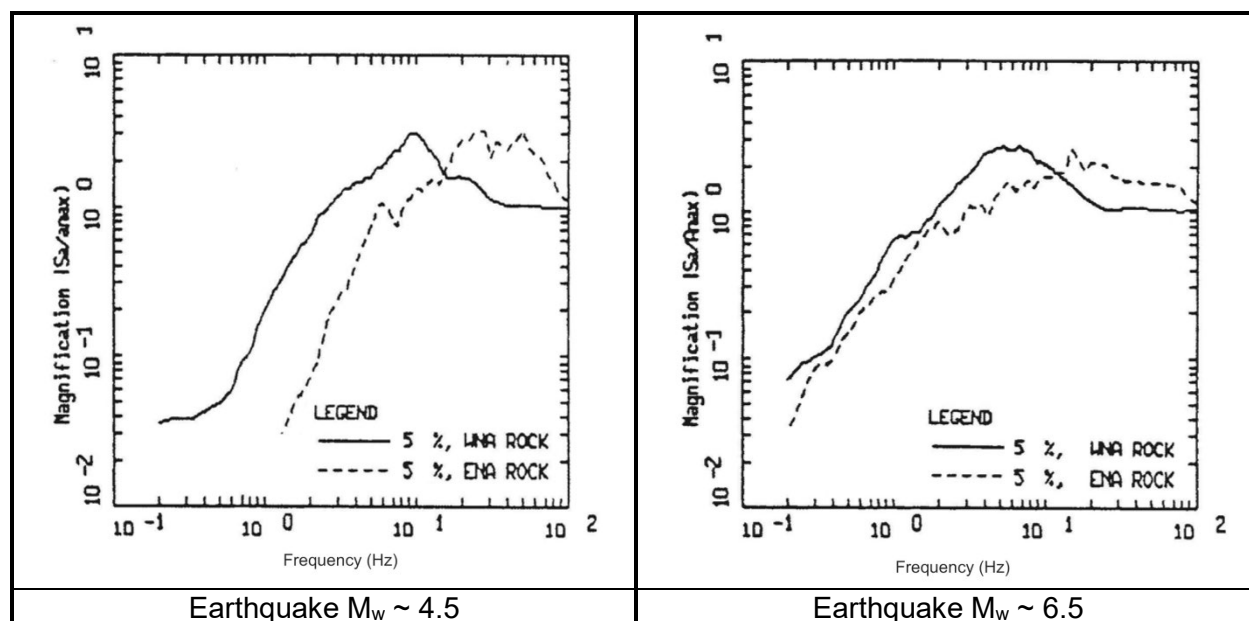


Figure 12-4, WNA / ENA Rock Effects on Normalized Response Spectra (Silva and Darragh, 1995)

12.7.2 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 along the “Fall Line” with the Columbia, SC metropolitan area being an example. 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 site-specific response study (Lester (2005)) of the Columbia, SC area compared spectral accelerations modeled at the 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 2 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 extended to the hard-rock outcropping conditions. The magnitude of the increase in spectral acceleration was dependent on the thickness of the graded weathered rock zone.

Based on this study (Lester (2005)) 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 Scenario_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 ($V_s = 2,500$ feet per second) and therefore the amplification observed from weathered rock thickness greater than 30 feet will not be as significant.

2. **Blue Ridge/Piedmont Unit with Weathered Rock Zone:** The 3-Point site response method can only be used if the weathered rock thickness ($2,500 \leq V_s \leq 8,200$ feet per second) is less than 30 feet thick. When performing site-specific response analyses in the Blue Ridge/Piedmont units with weathered rock zone ($2,500 \leq V_s \leq 8,200$ feet per second) 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.

12.7.3 **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 5 instrumented sites. Three of the sites were in the United States (California) and the other 2 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 T_N of the site as it softens (See Figure 12-1). 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 an SSRA is not performed and the simplified response methods that use the Seismic Hazard Mapping study (Section 12.8) 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 ($T_0 < 1.0$ 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 ($T_0 > 1.0$ sec), the design spectral accelerations may be unconservative due to the lengthening of the T_N of the site. For these types of structures with long-fundamental periods ($T_0 > 1.0$ sec), a site-specific seismic response analysis should be considered.

12.7.4 **Horizontal Ground Motion Response Spectra**

The Seismic Specs require 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 SEE. Bridges with an OC = I or II also require a structural evaluation using

the FEE only if the project site has the potential for SSL or slope instability at bridge abutments and no geotechnical mitigation is performed during the FEE. Seismic structural design shall be required, as required in the Seismic Specs, even if the displacement criteria established in GDM is met. Therefore, meeting the displacement criteria is not considered as geotechnical mitigation for meeting this design requirement.

The ADRS curves is determined using either the 3-Point method (Section 12.8) or the SSRA (Section 12.9) based on the selection criteria in Section 12.9.

ADRS curves described in Sections 12.8 and 12.9 are generated for the design earthquakes (SEE and/or FEE) as needed by the SEOR to perform a structural evaluation. However, a 2-level design approach (SEE and FEE) is required for all bridge embankments and all ERSs located within the limits of the bridge embankments. Therefore, the ADRS curve for both seismic events shall be developed and provided to the design team. ERSs located within the roadway embankment shall be designed for the SEE only; unless in the opinion of the design team a 2-level approach (i.e., designing for both FEE and SEE) should be considered. The ADRS curves are supplied to the SEOR 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 (see Figure 12-8 for format).

12.7.5 Vertical Ground Motion Response Spectra

Recent studies shown in Figure 12-5 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 $2/3$ ratio of vertical to horizontal ground motion response spectra may be conservative for T'_o longer than 0.2 seconds. For T'_o shorter than 0.2 seconds the ratio of vertical to horizontal ground motion response spectra may exceed the $2/3$ value and may be on the order of 1 to 1.5 times the horizontal for earthquakes with close source-to-site distances and T'_o of less than 0.1 seconds. Although the studies shown in Figure 12-5 are from ground motion data from the WUS, Chiou, Silva, and Power (2002) indicates that the ratios for the CEUS are not greatly different from the ratios in the WUS.

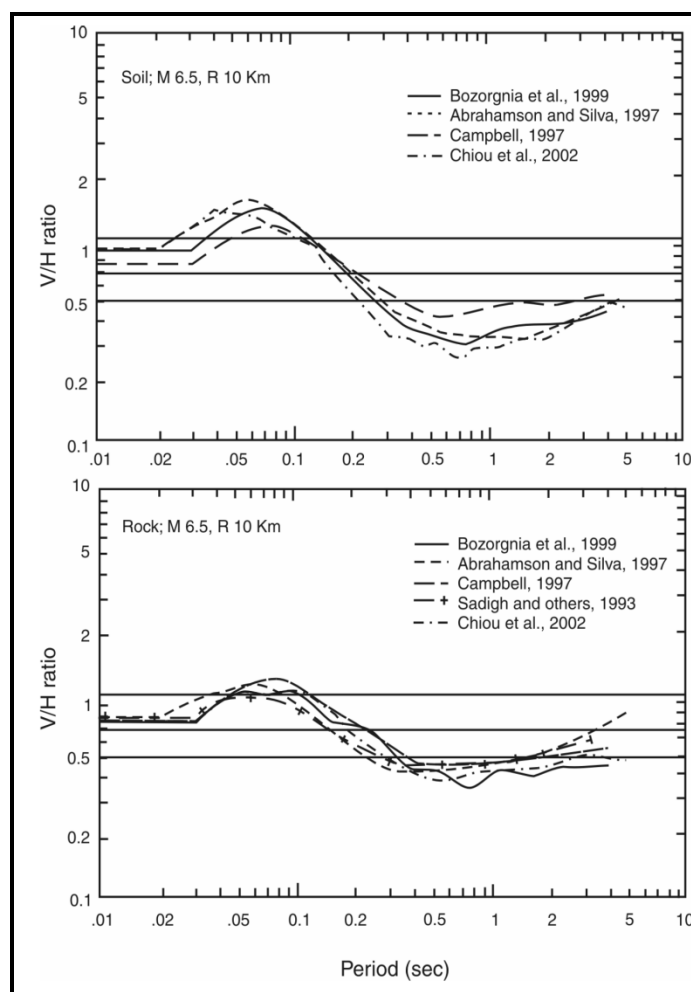


Figure 12-5, Vertical/Horizontal Spectral Ratios vs. Period (Buckle, et al. (2006))

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 Seismic Hazard Mapping study, Section 12.8, the 2/3 ratio of vertical-to-horizontal response spectra shall be used for bridges with T_0 of 0.2 seconds or longer. When the bridge's T_0 is less than 0.2 seconds, a site-specific vertical response spectrum using the results of recent studies such as those shown in Figure 12-5 should be used to develop the vertical ground motion response spectra.

12.8 SITE RESPONSE ANALYSIS USING SEISMIC HAZARD MAPPING STUDY

The results of the Seismic Hazard Mapping study (i.e., SCENARIO_PC (2006)) shall be used to develop the 3-Point ADRS curve. The 3-Point ADRS curve is anticipated to be used on all typical SCDOT bridges, except those sites meeting the Site Class F criteria provided in Section 12.4 or as determined by SCDOT. Non-typical bridges, sites with Site Class F soils and those bridges selected by SCDOT shall have an SSRA performed in accordance with Section 12.9. The following Sections describe the procedures for developing the site amplification factors, F_t that are required to develop the 3-Point ADRS curve.

12.8.1 ADRS Curves for FEE and SEE

As described in Chapter 11 there are 2 design seismic events used for evaluation of SCDOT structures, the FEE and the SEE. The PGA and spectral response accelerations, S_a , developed using Sections 12.8.2 and 12.8.3 will depend on which design earthquake is being analyzed and on the local site conditions. Selected locations within South Carolina have been used, where depending on the geology the site amplification factors, F_t , can be different (Figure 12-6). Figure 12-6, as well as indicated in Chapter 11, depicts South Carolina as divided between the Coastal Plain (SCCP) and the Piedmont ((SCP) areas outside of the Coastal Plain). This is a change from the previous site factors, where a single set of site amplification factors (PGA (F_{PGA}), short-period (F_a) and long-period (F_v)) were used for the entirety of South Carolina and with the sites being differentiated by Site Class.

Based on Andrus, et al. (2014), F_t was determined to vary greatly with the $V_{s,100}^*$ (the average shear wave velocity for the upper 100 feet of the site), specifically,

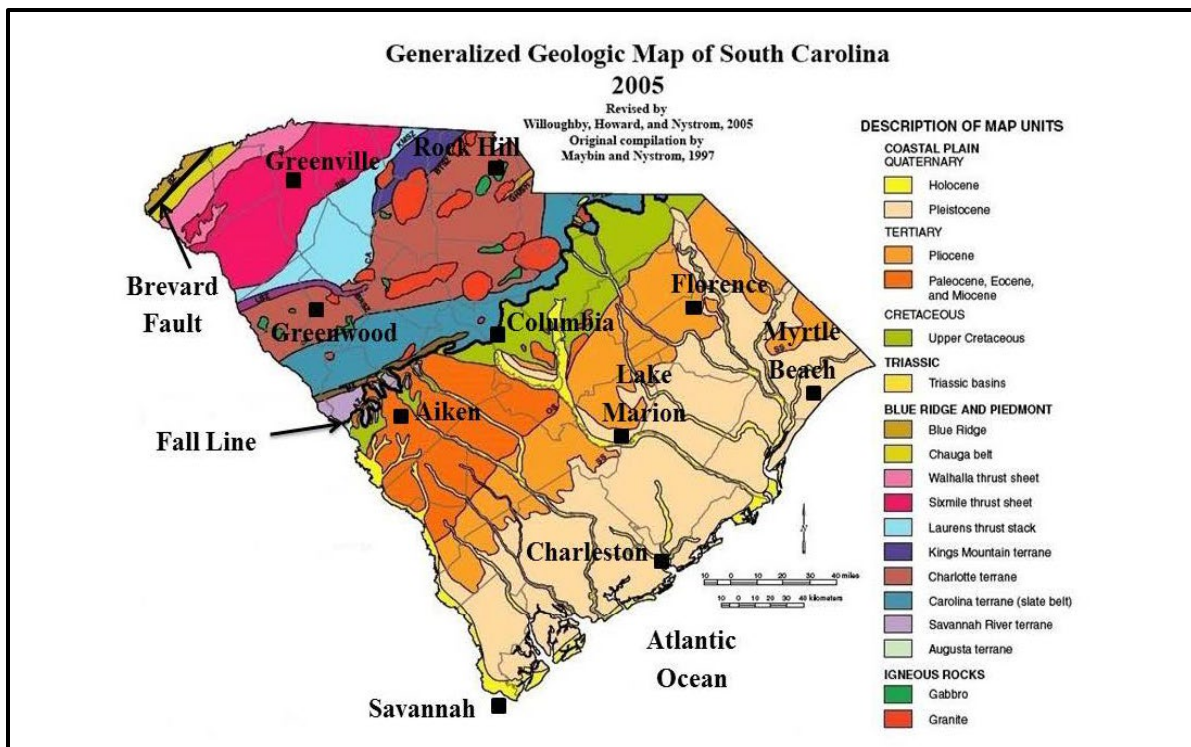
- An increasing trend in F_t as $V_{s,100}^*$ increased from a low value
- A zone of peak F_t values ($F_{P,t}$), depending on $V_{s,100}^*$ and PSA_{B-C}
- A decreasing trend in F_t as $V_{s,100}^*$ increases beyond the zone of F_P values

These trends are the same for both the Coastal Plain as well as the Piedmont. The F_t factors were determined for a range of spectral periods (t) and are referred to by the middle of the range periods as indicated in Table 12-6.

Table 12-5, Spectral Period Ranges and Designations

Spectral Period Range, t (sec)	Spectral Period Designation, t (sec)	Corresponding Pseudo-Acceleration, $PSA_{B-C,t}$ (g)	F_t Factor Designation
≤ 0.01	0.0	PGA_{B-C}	F_{PGA}
0.01 – 0.40	0.2	S_s	$F_{0.2} (F_a)$
0.41 – 0.80	0.6	$S_{0.6}$	$F_{0.6}$
0.81 – 1.20	1.0	$S_{1.0}$	$F_{1.0} (F_v)$
1.21 – 2.00	1.6	$S_{1.6}$	$F_{1.6}$
2.01 – 4.00	3.0	$S_{3.0}$	$F_{3.0}$

The ADRS curves generated using the Seismic Hazard Mapping Study will be based on a 5 percent viscous damping ratio since the pseudo spectral accelerations (PSA) have been generated for 5 percent damping.



Note: In the Columbia and Aiken areas, if depth to Weathered Hard Rock < 330 feet, Piedmont Factors shall be used.

Figure 12-6, Geologic Map Indicating Sites Used in Ground Response Analysis (Andrus, et al. (2014))

12.8.2 Geologically Realistic Local Site Effects – Coastal Plain

The F_t factors for the Coastal Plain are based on the soil column (model) beginning at the B-C Boundary (i.e., the depth where $V_{s,H}^*$ remains consistently more than 2,500 ft/sec (~760 m/sec)). For the reference models developed in Andrus, et al. (2014), the B-C Boundary (termed soft rock half space in Andrus, et al. (2014)) ranged from 450 ft (137 m) to 485 ft (148 m). The peak average shear wave velocity in the top 100 feet, $V_{s,100,P,t}^*$ and the corresponding peak site coefficient at a specific spectral period, $F_{P,t}$, can be determined using:

$$V_{s,100,P,t}^* = x_1 * \left(\frac{PSA_{B-C,t}}{1g}\right)^{x_2} * \left(\frac{T_m}{1s}\right)^{x_3} * K_{H2} \quad \text{Equation 12-13}$$

$$F_{P,t} = \left\{ x_4 * \left[e^{\left(\frac{x_5 * PSA_{B-C,t}}{1g}\right)} \right] * \left(\frac{T_m}{T_{330}}\right)^{x_6} + 1 \right\} * K_{H1} \quad \text{Equation 12-14}$$

Where,

$V_{s,100,P,t}^*$ = Weighted, average site stiffness in the top 100 feet corresponding to the peak site factor adjusted for d_{B-C} , ft/sec

$F_{P,t}$ = Peak F_t factor at a specific spectral period adjusted for d_{B-C}

t = Specific spectral period, second (see Table 12-6)

$x_{1 \text{ to } 6}$ = Regression coefficients (see Table 12-7)

d_{B-C} = Depth to B-C Boundary, ft

$PSA_{B-C,t}$ = Pseudo-acceleration at the B-C Boundary outcrop at a specific spectral period, from SCENARIO_PC (2006)

- T_m = Mean period of input rock motion, sec
- T_{330} = Period for the top 330 feet (100 meters) of the site, sec
- K_{H1} and K_{H2} = Adjustment factors for $d_{B-C} < 330$ feet, see Table 12-9

T_m may be estimated using Equation 12-15 and is applicable for those sites that are dominated by the Charleston seismic hazard zone (i.e., the deaggregation indicates that the dominate source of the seismic hazard is Charleston). T_{330} may be estimated using Equation 12-16.

$$T_m = 0.031 * \left(\frac{d_{HR}}{1000}\right) + 0.485 * \left(\frac{R}{1000}\right) + 0.233 \quad \text{Equation 12-15}$$

$$T_{330} = \frac{4*330}{V_{s,330}^*} = \frac{1,320}{V_{s,330}^*} \quad \text{Equation 12-16}$$

$$V_{s,330}^* = \frac{330}{\left(\frac{100}{V_{s,100}^*} + \frac{230}{V_{s,100-330}^*}\right)} \quad \text{Equation 12-17}$$

Where,

- d_{HR} = Depth to Hard Rock ($V_s \geq 11,000$ ft/sec) from SCENARIO_PC (2006), feet
- R = Site to source distance (see Chapter 11), miles
- $V_{s,100}^*$ = Weighted, average site stiffness in the top 100 feet at a specific site, ft/sec
- $V_{s,100-330}^*$ = Weighted, average site stiffness between the depths of 100 and 330 feet estimated on a regional basis, ft/sec
- $V_{s,330}^*$ = Weighted, average site stiffness for the top 330 feet combining the site stiffness at a specific site with the regional site stiffness below 100 feet, ft/sec

Typical values of $V_{s,330}^*$, $V_{s,100-330}^*$, T_{330} and T_m are provided in Table 12-8. As additional deep shear wave velocities are obtained (i.e., $V_{s,330}^*$), it may become possible to determine $V_{s,100-330}^*$. Until that time use Table 12-8 to determine $V_{s,330}^*$, $V_{s,100-330}^*$.

Table 12-6, Regression Coefficients for the Coastal Plain

PSA_{B-C,t}	x₁ (ft/sec)	x₂	x₃	x₄	x₅	x₆	a
PGA_{B-C}	846	0.222	-0.276	7.510	-4.394	1.614	- ¹
S_s	804	0.206	-0.141	7.305	-1.980	1.546	0.65
S_{0.6}	466	0.181	-0.721	10.691	-3.382	1.487	0.85
S_{1.0}	344	0.214	-0.867	4.929	-2.734	0.437	0.90
S_{1.6}	420	0.228	-0.647	3.477	-2.555	0.185	0.99
S_{3.0}	692	0.208	-0.036	0.720	-5.638	-0.860	0.99

¹Use Equation 12-18

Table 12-7, Typical Normalized Period Values by Region

Site Regions	$V_{s,330}^*$ (ft/sec)	$V_{s,100-330}^*$ (ft/sec)	T_{330} (sec)	T_m (sec)	T_m/T_{330}
Charleston	1,237	1,445	1.06	0.29	0.27
Savannah	1,237	1,445	1.06	0.40	0.38
Myrtle Beach	1,555	1,945	0.84	0.37	0.44
Columbia	1,381	1,620	0.95	0.29	0.30
Florence	1,381	1,620	0.95	0.30	0.32
Lake Marion	1,381	1,620	0.95	0.28	0.29
Aiken	1,299	1,370	1.01	0.31	0.31

Table 12-8, Adjustment Factors for $d_{B-C} < 330$ feet for the Coastal Plain

$PSA_{B-C,t}$	Adjustment Factor	Depth to B-C Boundary, d_{B-C} (feet)							
		1.5	5	16.5	33	65	100	165	≥ 330
PGA_{B-C}	K_{H1}	0.96	1.11	1.53	1.40	1.24	1.15	1.02	1.00
	K_{H2}	2.71	2.29	2.08	1.67	1.25	1.17	1.04	1.00
S_s	K_{H1}	0.77	0.90	1.23	1.55	1.35	1.23	1.10	1.00
	K_{H2}	2.71	2.29	1.88	1.50	1.25	1.04	1.02	1.00
$S_{0.6}$	K_{H1}	0.48	0.70	0.83	0.91	1.00	1.04	1.04	1.00
	K_{H2}	2.95	2.27	1.59	1.36	1.36	1.14	1.09	1.00
$S_{1.0}$	K_{H1}	0.46	0.73	0.80	0.84	0.88	0.92	0.96	1.00
	K_{H2}	2.86	2.14	1.52	1.43	1.29	1.19	1.05	1.00
$S_{1.6}$	K_{H1}	0.26	0.29	0.60	0.81	0.83	0.95	0.98	1.00
	K_{H2}	3.53	2.65	1.76	1.47	1.29	1.06	1.03	1.00
$S_{3.0}$	K_{H1}	0.37	0.41	0.46	0.61	0.69	0.78	0.89	1.00
	K_{H2}	5.36	4.02	2.68	1.88	1.52	1.34	1.07	1.00

As indicated previously, the F_t factor varies based on the shear wave velocity encountered at each site. A linear relationship for determining the F_t factor was developed by Andrus, et al. (2014) when $V_{s,100}^* < V_{s,100,P,t}^*$ and is applicable for all values of t ,

$$F_t = \left(\frac{F_{P,t}}{V_{s,100,P,t}^*} \right) * V_{s,100}^* \quad \text{Equation 12-18}$$

Where,

F_t = Amplification factor at a specific spectral period

$V_{s,100,P,t}^*$ = Weighted, average site stiffness in the top 100 feet corresponding to the peak site factor adjusted for d_{B-C} (Equation 12-13), ft/sec

$F_{P,t}$ = Peak F_t factor at a specific spectral period adjusted for d_{B-C} (Equation 12-14)

$V_{s,100}^*$ = Weighted, average site stiffness in the top 100 feet at a specific site, ft/sec

When $V_{s,100}^* \geq V_{s,100,P,t}^*$, the F_t factor for periods less than 0.2 seconds is expressed as a linear relationship. For periods greater than or equal 0.2 seconds the F_t factor is expressed as an exponential relationship. Both relationships were developed by Andrus, et al. (2014) and are provided below.

For $t < 0.2$ sec and $V_{s,100}^* \geq V_{s,100,P,t}^*$

$$F_t = \left[\frac{(F_{P,t}-1)(2,500-V_{s,100}^*)}{2,500-V_{s,100,P,t}^*} \right] + 1 \quad \text{Equation 12-19}$$

For $t \geq 0.2$ sec and $V_{s,100}^* \geq V_{s,100,P,t}^*$

$$F_t = a + b * e^{(c * V_{s,100}^*)} \quad \text{Equation 12-20}$$

Where,

F_t = Amplification factor at a specific spectral period

$V_{s,100,P,t}^*$ = Weighted, average site stiffness in the top 100 feet corresponding to the peak site factor adjusted for d_{B-C} (Equation 12-13), ft/sec

$F_{P,t}$ = Peak F_t factor at a specific spectral period adjusted for d_{B-C} (Equation 12-14)

$V_{s,100}^*$ = Weighted, average site stiffness in the top 100 feet at a specific site, ft/sec

a = Regression coefficient from Table 12-7

b = Regression coefficient determined from Equation 12-21

c = Regression coefficient determined from Equation 12-22, sec/ft

$$b = \frac{1-a}{e^{(2500*c)}} \quad \text{Equation 12-21}$$

$$c = \left(\frac{1}{2500-V_{s,100,P,t}^*} \right) * \ln \left(\frac{1-a}{F_{P,t}-a} \right) \quad \text{Equation 12-22}$$

If the project site is located within one of the Coastal Plain counties near the “Fall Line” (i.e., Aiken, Chesterfield, Kershaw, Lexington, or Richland Counties) and the depth to shallow weathered hard rock ($V_{s,100}^* \geq 8,200$ ft/sec) is less than 330 feet, then the F_t factors developed in Section 12.8.3 shall be used.

12.8.3 Geologically Realistic Local Site Effects – Outside the Coastal Plain

The F_t factors for the Piedmont, i.e., Geologically Realistic site conditions outside the Coastal Plain (see Figure 12-6) are based on the soil column (model) beginning at the Weathered Rock boundary (i.e., the depth where $V_{s,H}^*$ remains consistently greater than 8,200 ft/sec (2500 m/sec)). For the reference models developed in Andrus, et al. (2014), the Weathered Rock boundary ranged from 33 ft (10 m) to 100 ft (30 m). The peak F_t factor, F_P , and the peak average shear wave velocity for the top 100 ft, $V_{s,100,P}^*$, are determined using the following equations:

$$V_{s,100,P,t}^* = x_7 * \left(\frac{PSA_{WR,t}}{1g} \right)^{x_8} * \left(\frac{T_m}{1s} \right)^{x_9} * K_{H4} \quad \text{Equation 12-23}$$

$$F_{P,t} = \left\{ x_{10} * \left[e^{\left(\frac{x_{11} * PSA_{WR,t}}{1g} \right)} \right] + 1 \right\} * K_{H3} \quad \text{Equation 12-24}$$

Where,

- $V_{s,100,P,t}^*$ = Weighted, average site stiffness in the top 100 feet corresponding to the peak site factor adjusted for d_{WR} , feet per second
- $F_{P,t}$ = Peak F_t factor at a specific spectral period adjusted for d_{WR}
- D_{WR} = Depth to Weathered Rock ($V_s \geq 8,200$ feet per second)
- t = Specific spectral period, second (see Table 12-6)
- $x_{7 \text{ to } 11}$ = Regression coefficients (see Table 12-10)
- $PSA_{WR,t}$ = Pseudo-acceleration at the Weathered Rock (8,200 feet per second) outcrop at a specific spectral period, from SCENARIO_PC (2006)
- T_m = Mean period of input rock motion, second
- T_{330} = Period for the top 330 feet (100 meters) of the site, sec
- K_{H3} and K_{H4} = Adjustment factors for d_{WR} , see Table 12-12

T_m may be estimated using Equation 12-15 with depth to Hard Rock d_{HR} equal to 0 ($d_{HR} = 0$) and is applicable for those sites that are dominated by the Charleston seismic hazard zone (i.e., the deaggregation indicates that the source of the seismic event is Charleston). T_m for the SEE in the western Piedmont (Abbeville, Anderson, Greenville, Greenwood, Laurens, McCormick, Oconee and Pickens Counties) cannot be determined using Equation 12-15, since this area is dominated by different seismic hazard zone than the Charleston seismic hazard zone. For the western Piedmont, T_m shall be set as 0.37 sec ($T_m = 0.37$ sec) for the SEE condition.

Typical values of T_m are provided in Table 12-11. As additional deep shear wave velocities are obtained (i.e., $V_{s,330}^*$), it may become possible to determine $V_{s,100-330}^*$ and T_{330} may be estimated using the Equation 12-17.

Table 12-9, Regression Coefficients for the Piedmont

$PSA_{WR,t}$	x_7 (ft/sec)	x_8	x_9	x_{10}	x_{11}	a
PGA_{WR}	1,916	0.162	0.198	2.589	-3.772	⁻¹
S_s	1,765	0.180	0.184	2.420	-0.934	0.70
$S_{0.6}$	1,765	0.162	0.228	2.940	-2.653	0.99
$S_{1.0}$	1,227	0.090	0.333	1.489	-0.896	0.99
$S_{1.6}$	1,230	0.204	0.427	1.159	-1.423	0.99
$S_{3.0}$	695	0.208	-0.036	1.093	-4.480	0.99

¹Use Equation 12-25

Table 12-10, Typical Normalized Period Values by Region

Site Regions	T_m (sec)
Columbia	0.27
Rock Hill	0.28
Greenwood	0.35
Greenville	0.33

Table 12-11, Adjustment Factors for d_{WR} for the Piedmont

$PSA_{WR,t}$	Adjustment Factor	Depth to Weathered Rock Boundary, d_{WR} (feet)						
		16.5	33	66	100	131	165	330
PGA_{WR}	K_{H3}	0.35	0.37	1.00	1.00	0.96	0.89	0.78
	K_{H4}	7.83	7.33	1.67	1.00	0.97	0.93	0.77
S_s	K_{H3}	0.34	0.37	1.13	1.00	0.94	0.87	0.79
	K_{H4}	7.03	6.25	1.41	1.00	0.97	0.94	0.78
$S_{0.6}$	K_{H3}	0.30	0.32	0.62	1.00	1.04	1.05	1.18
	K_{H4}	9.69	9.39	2.86	1.00	0.97	0.94	0.79
$S_{1.0}$	K_{H3}	0.35	0.36	0.45	1.00	1.15	1.19	1.25
	K_{H4}	12.63	12.11	3.79	1.00	0.95	0.89	0.63
$S_{1.6}$	K_{H3}	0.59	0.61	0.77	1.00	1.03	1.03	1.10
	K_{H4}	12.00	11.00	3.60	1.00	0.95	0.90	0.65
$S_{3.0}$	K_{H3}	0.78	0.78	0.91	1.00	1.03	1.03	1.11
	K_{H4}	13.16	11.58	3.79	1.00	0.89	0.79	0.26

As indicated previously, the F_t factor varies based on the shear wave velocity encountered at each site. A linear relationship for determining the F_t factor was developed by Andrus, et al. (2014) when $V_{s,100}^* < V_{s,100,P,t}^*$ and is applicable for all values of t ,

$$F_t = \left(\frac{F_{P,t}}{V_{s,100,P,t}^*} \right) * V_{s,100}^* \quad \text{Equation 12-25}$$

Where,

F_t = Amplification factor at a specific spectral period

$V_{s,100,P,t}^*$ = Weighted, average site stiffness in the top 100 feet corresponding to the peak site factor adjusted for d_{HR} (Equation 12-23), ft/sec

$F_{P,t}$ = Peak F_t factor at a specific spectral period adjusted for d_{HR} (Equation 12-24)

$V_{s,100}^*$ = Weighted, average site stiffness in the top 100 feet at a specific site, ft/sec

When $V_{s,100}^* \geq V_{s,100,P,t}^*$, the F_t factor for periods less than 0.2 seconds is expressed as a linear relationship. For periods greater than or equal 0.2 seconds the F_t factor is expressed as an exponential relationship. Both relationships were developed by Andrus, et al. (2014) and are provided below,

For $t < 0.2$ sec and $V_{s,100}^* \geq V_{s,100,P,t}^*$

$$F_t = \left[\frac{(F_{P,t}-1)*(8,200-V_{s,100}^*)}{8,200-V_{s,100,P,t}^*} \right] + 1 \quad \text{Equation 12-26}$$

For $t \geq 0.2$ sec and $V_{s,100}^* \geq V_{s,100,P,t}^*$

$$F_t = a + b * e^{(c*V_{s,100}^*)} \quad \text{Equation 12-27}$$

Where,

F_t = Amplification factor at a specific spectral period

$V_{s,100,P,t}^*$ = Weighted, average site stiffness in the top 100 feet corresponding to the peak site factor adjusted for d_{HR} (Equation 12-23), ft/sec

$F_{P,t}$ = Peak F factor at a specific spectral period adjusted for d_{WR} (Equation 12-24)

$V_{s,100}^*$ = Weighted, average site stiffness in the top 100 feet at a specific site, ft/sec

a = Regression coefficient from Table 12-10

b = Regression coefficient determined from Equation 12-28

c = Regression coefficient determined from Equation 12-29, sec/ft

$$b = \frac{1-a}{e^{(8,200*c)}} \quad \text{Equation 12-28}$$

$$c = \left(\frac{1}{8,200 - V_{s,100,P,t}^*} \right) \ln \left(\frac{1-a}{F_{P,t} - a} \right) \quad \text{Equation 12-29}$$

12.8.4 Hard Rock Local Site Effects

When Hard Rock geologic conditions (i.e., $V_s \geq 11,500$ feet per second) occur within 100 feet of the ground surface or at the ground surface, the output from SCENARIO_PC (2006) is modified by an F-factor equal to 0.8.

12.8.5 Local Site Effects on Spectral Response Accelerations

The PSA values, generated from the Seismic Hazard Mapping study, as indicated in Section 12.6 and Chapter 11 at the B-C Boundary for the Coastal Plain and the weathered rock for the Piedmont, are termed the UHS. The PGA_{B-C} or PGA_{WR} , S_S and S_1 shall be obtained for the appropriate design earthquake (FEE or SEE) being analyzed. The PGA , S_{DS} and S_{D1} at the ground surface shall be determined by adjusting the PGA_{B-C} or PGA_{WR} , S_S and S_1 using the F_t factors developed in the previous Sections based on Geologically Realistic conditions of the site (i.e., Coastal Plain or outside of the Coastal Plain) using the following equations.

$$PGA = F_{PGA} * PGA_{B-C} \text{ or } F_{PGA} * PGA_{WR} \quad \text{Equation 12-30}$$

$$S_{DS} = F_a * S_S \quad \text{Equation 12-31}$$

$$S_{D1} = F_v * S_1 \quad \text{Equation 12-32}$$

Where:

PGA_{B-C} = Mapped peak ground acceleration at the B-C boundary outcrop (period, $t = 0.0$ sec)

PGA_{WR} = Mapped peak ground acceleration at the Weathered Rock outcrop (period, $t = 0.0$ sec)

PGA = Peak ground acceleration at the original ground surface (period, $t = 0.0$ sec) adjusted for local site conditions

S_S = The mapped spectral acceleration for the short-period (0.2-second) as determined in Section 12.8 and Chapter 11 at the B-C boundary or Weathered Rock outcrop

S_{DS} = Design short-period (0.2-second = 5 Hz) spectral response acceleration parameter

S_1 = The mapped spectral acceleration for the one second period as determined in Section 12.8 and Chapter 11 at the B-C boundary or Weathered Rock outcrop

S_{D1} = Design long-period (1.0 second = 1 Hz) spectral response acceleration parameter

F_{PGA} = Site amplification factors determined in the preceding Sections

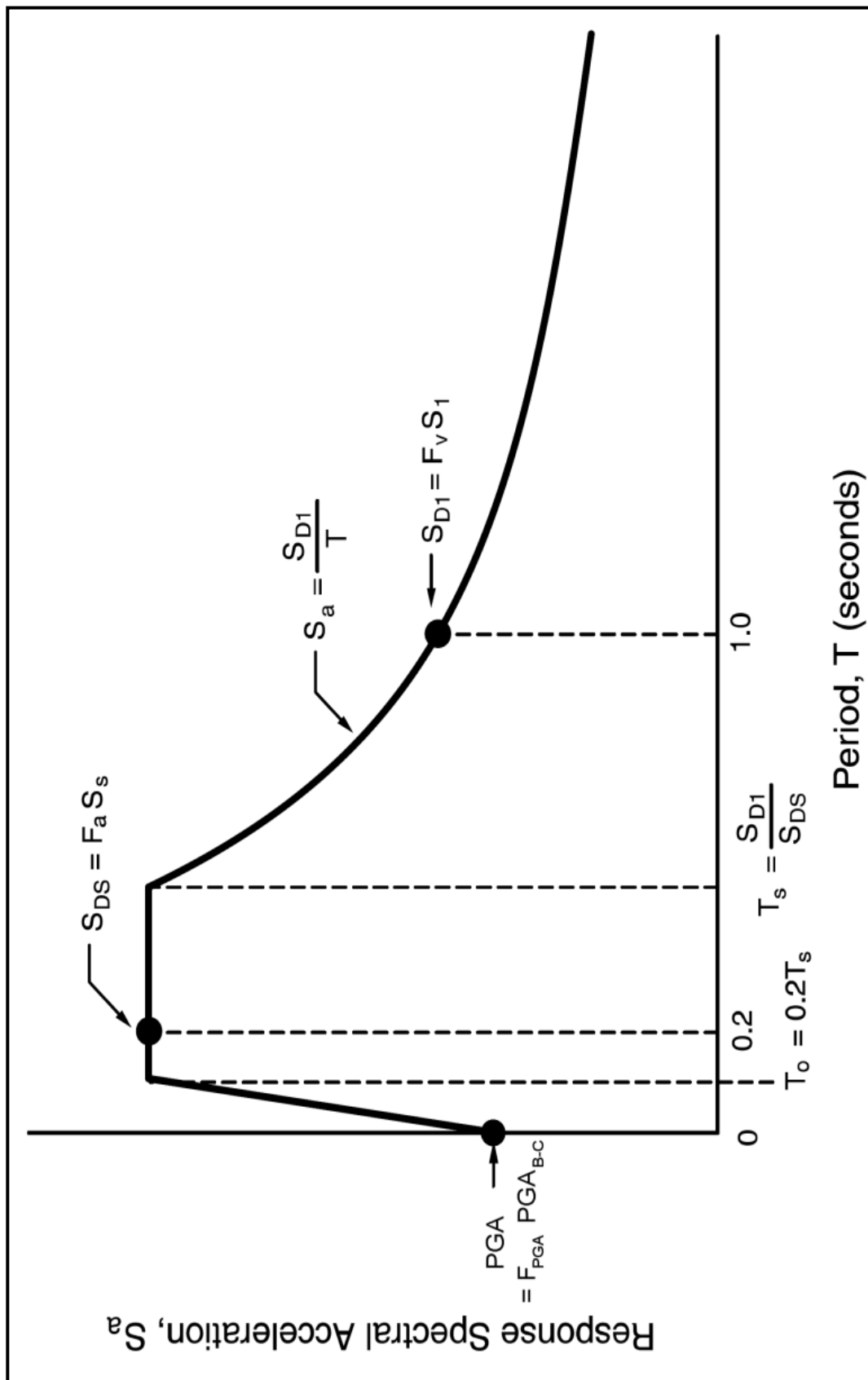
$F_a = F_{0.2}$ = Site amplification factors determined in the preceding Sections

$F_v = F_{1.0}$ = Site amplification factors determined in the preceding Sections

Use PGA , S_s and S_1 at the Hard Rock conditions as developed by SCENARIO _PC 2006) multiplied by 0.80 as the design values

12.8.6 3-Point Acceleration Design Response Spectrum

The 3-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 3-Point method has been shown by Power, et al. (1997, 1999) to be unconservative in the CEUS for periods between 1.0 second and 3.0 seconds, and a Site Class B (Rock). When the T_0 is less than 0.2 seconds or greater than 3.0 seconds, an SSRA as described in Section 12.9 may be required. Therefore, the 3-Point method shall be limited to T_0 equal to or less than 3.0 seconds (i.e., $T_0 \leq 3.0$ seconds) as indicated in Step 7 of Table 12-13. The Multi-Point method shall be used to evaluate the reasonableness of the 3-Point ADRS Curve as discussed in Section 12.8.6. Guidelines for constructing the 3-Point ADRS Curve are illustrated in Figure 12-7 and step-by-step instructions are provided in Table 12-13.



Note: PGA_{WR} may be substituted for PGA_{B-C}

Figure 12-7, 3-Point ADRS Curve Construction

Table 12-12, 3-Point ADRS Construction Procedures

Step	Procedure Description
1	The design short-period acceleration, S_{DS} , at period, $T = 0.2$ second and the design long-period acceleration, S_{D1} , at period, $T = 1.0$ second are computed using Section 12.8.4.
2	<p>Period markers T_o and T_s used in constructing the ADRS curves are determined using the following equations.</p> $T_s = \frac{S_{D1}}{S_{DS}} \quad \text{Equation 12-33}$ $T_o = 0.20 * T_s \quad \text{Equation 12-34}$ <p>Where S_{DS} and S_{D1} are obtained in Step 1.</p>
3	The PGA at the original ground surface at period, $T=0.0$ second is computed using Section 12.8.4.
4	<p>The design spectral response acceleration S_a for periods, $T \leq T_o$, is computed by the following equation.</p> $S_a = PGA + \left[(S_{DS} - PGA) * \left(\frac{T}{T_o} \right) \right] \quad \text{Equation 12-35}$ <p>Where, S_{DS} is obtained in Step 1, T_o is obtained in Step 2, and PGA is obtained in Step 3.</p>
5	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.
6	<p>The design spectral response acceleration, S_a, for periods, $T_s < T \leq 3.0$ seconds, is computed by the following equation.</p> $S_a = \frac{S_{D1}}{T} \quad \text{Equation 12-36}$ <p>Where, S_{D1} is obtained in Step 1.</p>
7	<p>The 3-Point ADRS curve shall include the following items:</p> <ul style="list-style-type: none"> • 3-Point ADRS curve (both FEE and SEE as required) • Table of smoothed ADRS data values (T and S_a) • Provide the design spectral response parameters PGA, S_{DS}, S_{D1}; period markers T_o and T_s; M_w and R; PGV; D_{a5-95}; PGV; T'_o; T_0; $V^*_{s,H}$; H; and T_{NH}. An example of the information required is shown in Figure 12-8.

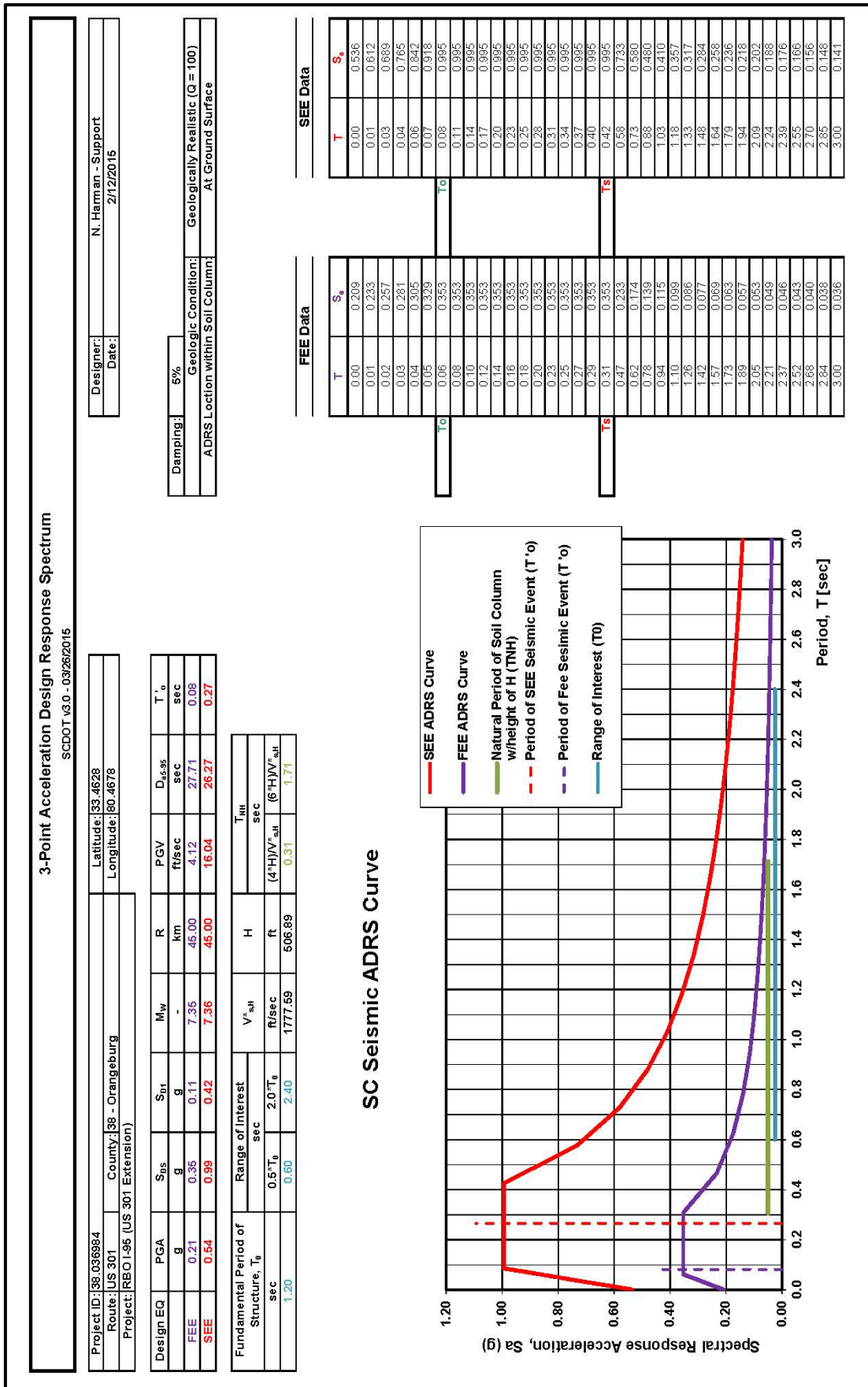


Figure 12-8, 3-Point ADRS Curve

12.8.7 Multi-Point Acceleration Design Response Spectrum

The Multi-point method of constructing an ADRS curve shall be used to check the reasonableness of the 3-Point ADRS curve. This is accomplished by first constructing the 3-Point ADRS curve and then overlaying on the same graph the Multi-point ADRS values as shown in Figure 12-9. The GEOR should be aware that Power, et al. (1999) have found that the Multi-point method may give ambiguous results for structures on sites other than rock ($V_s > 2,500$ ft/sec). This is due to the Multi-point method using the short period (0.2 seconds) site factor F_a ($F_{0.2}$) 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 ($F_{1.0}$), for all periods greater than or equal to 1.0 seconds to compute the acceleration response spectrum. Because of this ambiguous result Andrus et al. (2014) provided a method to develop F factors at other periods. The procedures provided in the previous Sections shall be used to develop the Multi-point curve. Andrus et al. (2014) recommends the use of the Multi-point method when $V_{s,100}^* < 660$ ft/sec. However, the Multi-point method shall be used for all ranges of $V_{s,100}^* < 2,500$ ft/sec. Since the Multi-point method is only used to check the reasonableness of the 3-Point ADRS curve for sites with $V_{s,100}^* < 2,500$ ft/sec this procedure should be adequate. Guidelines for constructing the Multi-Point ADRS curve are provided in Table 12-14.

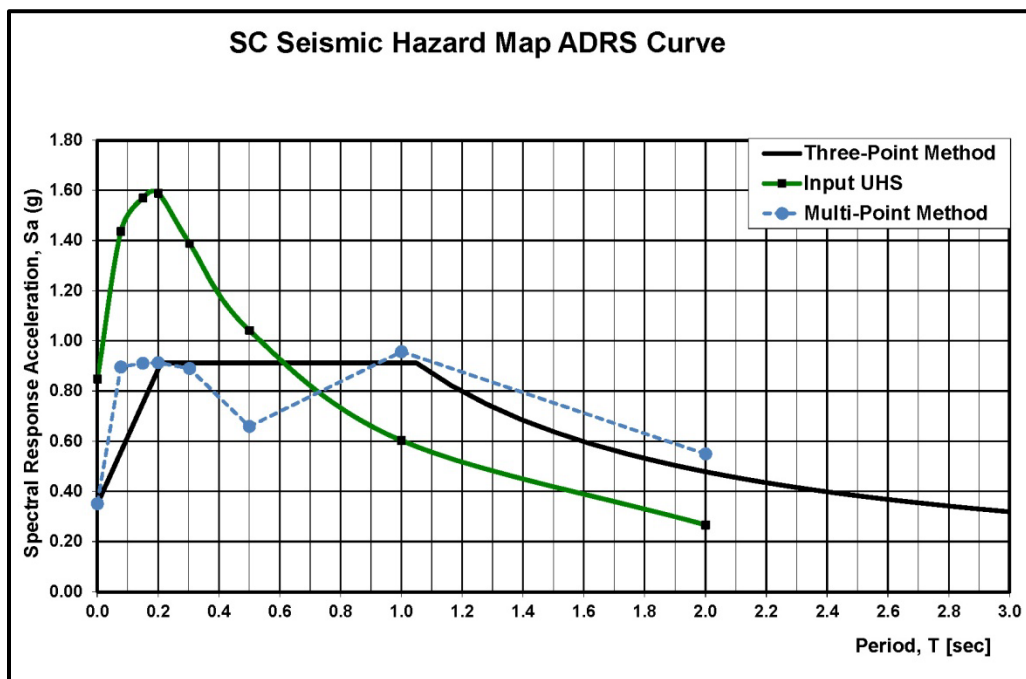


Figure 12-9, 3-Point/Multi-Point ADRS

Table 12-13, Multi-Point ADRS Construction Procedure

Step	Procedure Description
1	The FEE or SEE mapped pseudo spectral accelerations at the B-C boundary (PSA_{B-C}) for periods, $T = 2.0$ sec (0.5Hz), 1.0 sec (1.0Hz), 0.303 sec (3.3Hz), 0.20 sec (5Hz), 0.15 sec (6.7Hz), 0.08 sec (13Hz) and PGA (PGA_{B-C}) are obtained from the SC Seismic Hazard map as indicated in Section 12.6 and Chapter 11.
2	The PGA, S_{DS} , S_{D1} are computed using Section 12.8.4.
3	The design spectral response acceleration, S_a , for periods, $0.01 \leq T \leq 0.40$ second is computed using the following equation. <p style="text-align: center;">$S_a = F_s * S_s \quad \text{Equation 12-37}$</p> <p>Where S_s includes PSA_{B-C} for periods, $T = 0.08$ sec (13Hz), 0.15 sec (6.7Hz), 0.20 sec (5Hz) and 0.303 sec (3.3Hz) from Step 1. The site factor F_s is obtained as indicated in Sections 12.8.2 (Coastal Plain) and 12.8.3 (Piedmont).</p>
4	The design spectral response acceleration, S_a , for periods, $0.41 \leq T \leq 0.80$ second is computed using the following equation. <p style="text-align: center;">$S_a = F_{0.6} * S_{0.6} \quad \text{Equation 12-38}$</p> <p>Where $S_{0.6}$ is the PSA_{B-C} for periods, $T = 0.5$ sec (2Hz) from Step 1. The site factor $F_{0.6}$ is obtained as indicated in Sections 12.8.2 (Coastal Plain) and 12.8.3 (Piedmont).</p>
5	The design spectral response acceleration, S_a , for periods, $0.81 \leq T \leq 1.20$ second is computed using the following equation. <p style="text-align: center;">$S_a = F_{1.0} * S_{1.0} \quad \text{Equation 12-39}$</p> <p>Where $S_{1.0}$ is the PSA_{B-C} for 1.0 sec (1.0Hz). The site factor $F_{1.0}$ is obtained as indicated in Sections 12.8.2 (Coastal Plain) and 12.8.3 (Piedmont).</p>
6	The design spectral response acceleration, S_a , for periods, $1.21 \leq T \leq 2.00$ second is computed using the following equation. <p style="text-align: center;">$S_a = F_{1.6} * S_{1.6} \quad \text{Equation 12-40}$</p> <p>Where $S_{1.6}$ is the PSA_{B-C} for 2.0 sec (0.5Hz). The site factor $F_{1.6}$ is obtained as indicated in Sections 12.8.2 (Coastal Plain) and 12.8.3 (Piedmont).</p>
7	The design spectral response acceleration, S_a , for periods, $2.01 \leq T \leq 4.00$ second is computed using the following equation. <p style="text-align: center;">$S_a = F_{3.0} * S_{3.0} \quad \text{Equation 12-41}$</p> <p>Currently, $S_{3.0}$ is not determined; however, in the future this value may be added to the ADRS curve development. The site factor $F_{3.0}$ is obtained as indicated in Sections 12.8.2 (Coastal Plain) and 12.8.3 (Piedmont). For periods greater than 4.01 seconds a site-specific response analysis shall be required.</p>

Note: This Table indicates B-C Boundary conditions; however, WR conditions, PSA_{WR} may be substituted for PSA_{B-C}

After the Multi-point horizontal ADRS curve has been constructed, the following should be checked by both the SEOR and the GEOR to see if the 3-Point ADRS curve is either underestimating spectral accelerations or not representative of the acceleration response spectrum. The SEOR will provide the fundamental periods of vibration that are important to the structural response and the GEOR will compare this value to the Multi-point spectral acceleration curve.

- 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 to the 0.10 sec period to assure that the short-period response is not underestimated.
- Check to see if the general trend of the 3-Point ADRS curve is similar to the Multi-point ADRS curve. If the fundamental period of the structure is in the range of longer periods the spectral accelerations will be significantly underestimated using the 3-Point ADRS.

If discrepancies between the 3-Point method and the Multi-point method have the potential to significantly underestimate the spectral response, the OES/GDS must be contacted. The OES/GDS will either approve modifications to the 3-Point ADRS curve or require a site-specific response analysis.

12.8.8 ADRS Evaluation using Seismic Hazard Mapping Study

Even though ADRS determination using the Seismic Hazard Mapping study is relatively straight forward, a series of checks are necessary to ensure its appropriateness. This involves using the 3-Point method as the basis of developing the ADRS curve and the Multi-point method to confirm its validity. A decision flow chart is provided in Appendix J to assist the designer with developing the ADRS curve based on the Seismic Hazard Mapping Study.

12.8.9 Damping Modifications of ADRS Curves

The ADRS curves developed using the Seismic Hazard Mapping Study is 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 12-15. For spectra constructed using the 3-Point method, the factors for periods of 0.20 sec and 1.0 sec can be used.

**Table 12-14, Damping Adjustment Factors
(Newmark and Hall (1982) and Idriss (1990))**

Period (seconds)	Ratio of Response Spectral Acceleration for Damping Ratio λ to Response Spectral Acceleration for $\xi_{\text{eff}} = 5\%$		
	$\lambda_{\text{eff}} = 2\%$	$\lambda_{\text{eff}} = 7\%$	$\lambda_{\text{eff}} = 10\%$
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

12.9 SITE-SPECIFIC RESPONSE ANALYSIS

The SSRA requirements in this Section apply only to “typical” bridges as defined by the Seismic Specs. Similarly to the 3-Point method ADRS curve development, all SSRAs shall be performed by the OES/GDS. The SSRA shall be considered 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 12.8.6)
- T_{NH} and T_o intersect on the 3-Point ADRS Curve (Figure 12-8)
- As required by SCDOT

In addition, a SSRA may be required for a structure meeting the following criteria:

- T_o is less than 0.2 seconds or more than 3.0 seconds (i.e., $T_o < 0.2$ sec or $T_o > 3.0$ sec)

As required in Chapter 11, a minimum of 7 time histories (synthetic or “real”) shall be required. The synthetic time histories shall be developed as required in Chapter 11. In addition, the “real” time histories shall be selected as required in Chapter 11. It is noted that prior to performing a SSRA a 3-Point ADRS is required. The 3-Point curve shall be used for comparison purposes with the SSRA as required in Section 12.9.4.

12.9.1 Equivalent-Linear 1-Dimensional Site-Specific Response

An equivalent-linear 1-dimensional SSRA shall be performed using SHAKE2000 or other computer software that is based on the SHAKE2000 computational model. The SHAKE2000 computer program models a soil column with horizontal layered soil deposits overlying a uniform visco-elastic half space. The SHAKE2000 computer program is based on the original SHAKE program was developed by Schnabel, Lysmer, and Seed (1972), and updated by Idriss and Sun (1992) to SHAKE91. SHAKE91 was updated by Ordóñez (2011) with SHAKEDIT added as a pre- and post-processor to SHAKE91. The computer program DeepSoil (Hashash (2012)) has been developed specifically for the CEUS and performs the equivalent linear analysis similar to SHAKE2000. The OES/GDS shall approve in writing the use of software other than SHAKE2000

or DeepSoil. The software must be nationally recognized in the United States as SHAKE2000 type software.

For most projects and site conditions, the SHAKE2000 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 1-dimensional site-specific response analysis may be required. Situations where an equivalent-linear 1-dimensional site-specific response analysis (SHAKE2000) method has been shown to be unreliable are listed below:

- When the PGA at the ground surface is 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 ($S_t > 8$).

12.9.2 Non-Linear 1-Dimensional Site-Specific Response

A non-linear 1-dimensional analysis shall be required when a SSRA is required and the PGA_{B-C} is greater than 0.3 g ($PGA_{B-C} > 0.3$ g). Both total and effective stress analyses shall be performed. It is noted that the pore water pressure generation model shall be matched as closely as can be expected to the soils on the project site. Guidance in using non-linear site response analysis procedures can be obtained from Stewart, et al. (2008). One-dimensional non-linear site response analyses shall be performed using approved computer software such as DMOD2000 (Matasović and Ordóñez (2011)) 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 DESRA-MUSC (Qiu, (1998)), and DeepSoil (Hashash (2012)) have been developed that modify and improve the accuracy of the constitutive soil models originally developed. Authorized software used to perform 1-dimensional non-linear site-specific response analysis must be based on DMOD2000 (Matasović and Ordóñez (2011)), DeepSoil (Hashash (2012)) or equivalent. The OES/GDS shall approve in writing the use of software other than DMOD2000 or DeepSoil. The software must be nationally recognized in the United States. Nonlinear site response codes such as DMOD2000 have issues estimating both small and large strain damping (Phillips and Hashash (2009)). DeepSoil has theoretical improvements on this matter and therefore better accuracy in computed responses is expected from this software.

12.9.3 Site-Specific Response Analysis Methodology

A 1-dimensional soil column model is needed when performing a SSRA using either the equivalent-linear or non-linear methods. The soil column extends from either the bedrock or the Geologically Realistic site condition (B-C Boundary or Weathered Rock Boundary) to the location where the ground motion transmits the ground shaking energy to the structure being designed, typically the ground surface.

When performing either an equivalent-linear or non-linear 1-dimensional site-specific response analysis, the soil layers in the 1-dimensional column are characterized by the layer thickness, H_i ; soil description including classification testing and geologic age; total unit weight (γ_T); and, Shear Wave Velocity (V_s). The development of the 1-dimensional soil column for a project site may require making several assumptions as to the selection of layer thicknesses and soil properties. Individual layer thicknesses should be no greater than:

$$H_i = \frac{V_{s,i}}{100} \quad \text{Equation 12-42}$$

Where,

$V_{s,i}$ = Shear wave velocity for each layer, ft/sec

H_i = Thickness of each individual layer, feet

100 = Constant representing 4 times the maximum frequency of the individual layer, assuming the maximum frequency is 25 Hz

In addition, a layer shall be placed at the ground water table used in the model; i.e., the ground water table shall be located at the interface between 2 soil layers. The soil parameters required are described in Chapter 7. The soil column model should be prepared in tabular form similar to Table 12-16. An equivalent linear code uses a constant number for both shear modulus and damping ratio for the entire excitation period while a non-linear code picks different numbers for both shear modulus and damping ratio corresponding to the varying shear strain during excitation.

Table 12-15, One-Dimensional Soil Column Model

Geologic Time	Layer No.	Layer Thickness, H_i	Soil Formation	Soil Description (USCS)	PI	FC	Total Unit Weight, γ_T	Shear Wave Velocity, $V_{s,i}$
Quaternary	1							
	2							
Neogene	3							
Paleogene	4							
Cretaceous	5							
	6							
Bed Rock	i							

The OES/GDS shall perform a sensitivity analysis on the 1-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.
- Variation in soil properties to account for effects of ground improvement, specifically, if deep soil mixing or some form of grouting is used to bind the soil grains together.

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. The ARS curve developed from the baseline model (i.e., the base model used in the sensitivity analysis) shall be given no less than 5 percent weight nor more than 10 percent weight over the other ARS curves developed during the sensitivity analyses. A single recommended site-specific horizontal ARS curve should be superimposed on the graph to develop a site-specific ADRS curve. Since 7 ground motions will be used, the arithmetic mean of the ARSs shall be used to develop the site-specific ADRS curve. The method of selecting the recommended site-specific ARS curve shall be documented in the report. The sensitivity analysis will be required for each ground motion developed for the project site.

When performing a non-linear 1-dimensional 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 1-dimensional site-specific response analysis.

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

12.9.4 Site-Specific Horizontal ADRS Curve

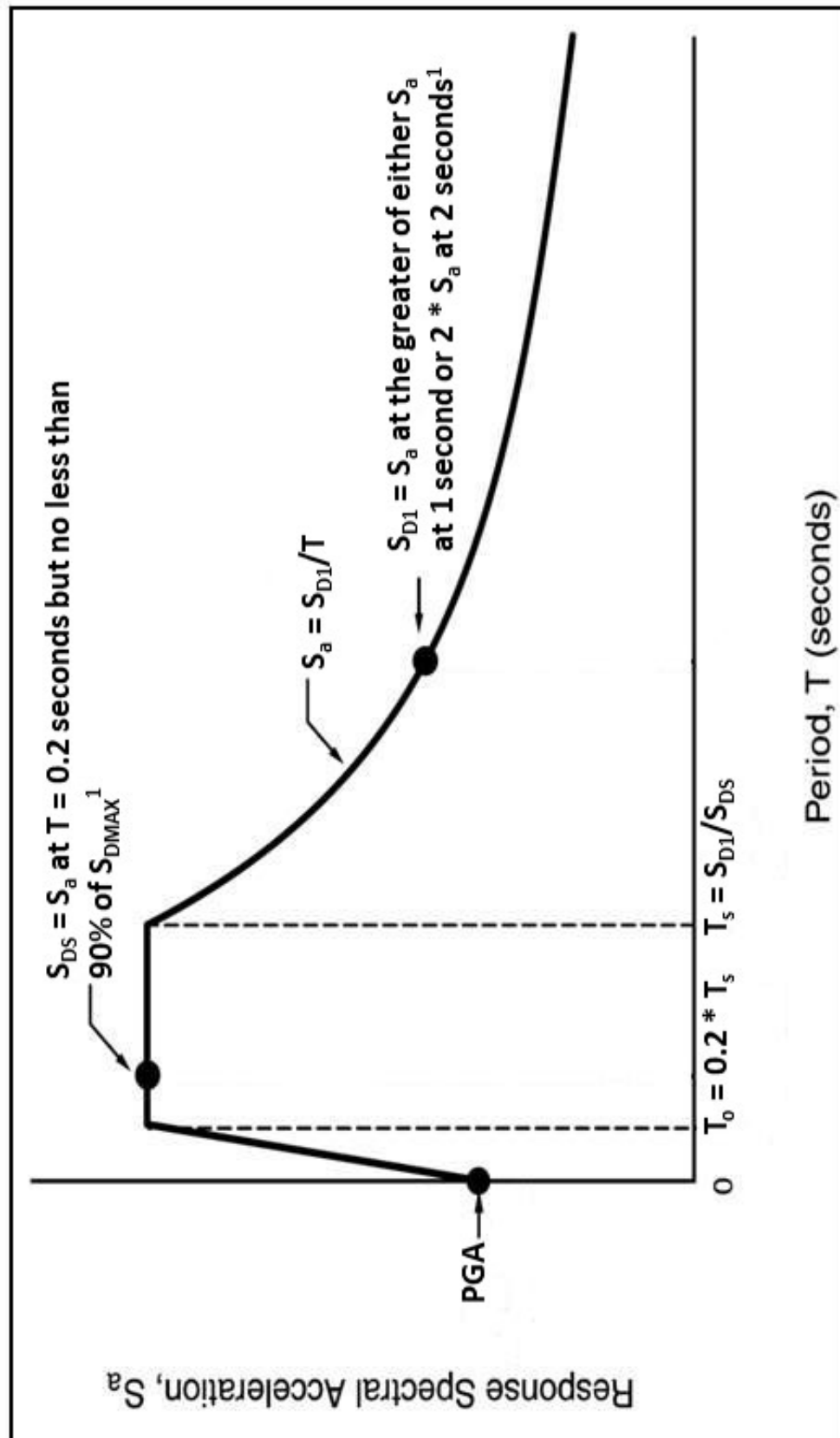
The development of the recommended site-specific ADRS shall be based on results of the SSRA (Sections 12.9.1 or 12.9.2) and shall be developed at the existing ground surface unless the requirements of Section 12.5 are met (i.e., the SEOR requests the development of Site-Specific ADRS curve at a different depth than the ground surface). The Site-Specific ADRS curve shall 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 (see Section 12.8.8). When the Site-Specific ADRS curve has spectral accelerations in the period range of greatest significance to the structural response (typically 0.5 to 2.0 seconds; for T_0 equal to 1.0 second, where T_0 is the fundamental period of the bridge or structure) are between the 3-Point ADRS curve and 70 percent of the 3-Point ADRS curve, the Site-Specific ADRS curve shall be used. If any point of the Site-Specific ADRS curve is less than 70 percent of the spectral accelerations computed using the 3-Point method, the OES/GDS shall be consulted to determine if the 70 percent of the 3-point curve will be used or if the spectral accelerations less than the 70 percent criterion can be used or if an independent third-party review (Peer Review) of the ADRS curve by an individual with the expertise in the evaluation of ground motions is to be undertaken. The Peer Review shall be conducted by an individual who has a minimum of 10 years' experience in geotechnical seismic design and who shall have conducted a minimum of 7 SSRAs as the lead designer. If a non-linear analysis is performed, the PEER Reviewer shall have conducted at least 3 non-linear site response analyses. The 3 non-linear analyses may be included in the 7 site-specific response analyses. In addition, the Peer Reviewer shall be licensed as either an engineer (PE) or geologist (PG) pursuant to the laws of South Carolina. Rarely, a Site-Specific ADRS has results that are greater than the 3-Point ADRS, when this occurs the larger Site-Specific ADRS

shall be used in-lieu of the 3-Point ADRS curve. Further it is possible that for a given period range the Site-Specific ADRS may be greater than the 3-Point ADRS, but less than 70 percent of the 3-Point ADRS over another range of periods. Therefore Site-Specific ADRS curves that are greater than the 3-Point ADRS curve shall be PEER reviewed with the PEER reviewer meeting the criteria previously stated.

A smoothed ADRS curve shall be superimposed (see Figure 12-11) over the recommended site-specific acceleration response spectrum generated from SSRA (Sections 12.9.1 or 12.9.2). The steps to develop the smoothed ADRS curve shall be based on Table 12-17 and Figure 12-10.

Table 12-16, Site-Specific ADRS Construction Procedures

Step	Procedure Description
1	The design short-period acceleration, S_{DS} , shall be the S_a at $T = 0.20$ seconds but shall not be less than 90 percent of the maximum design spectral response acceleration, S_{DMax} , at any period greater than 0.20 seconds.
2	The design long-period acceleration, S_{D1} , shall be the greater of either the S_a at $T = 1.0$ second or twice the S_a at $T = 2.0$ seconds.
3	Period markers T_o and T_s used in constructing the Site-Specific ADRS curves are determined using the following equations. $T_s = \frac{S_{D1}}{S_{DS}} \quad \text{Equation 12-43}$ $T_o = 0.20 * T_s \quad \text{Equation 12-44}$ <p>Where S_{DS} and S_{D1} are obtained in Steps 1 and 2.</p>
4	The PGA at the original ground surface shall be determined, $T=0.0$ second.
5	The design spectral response acceleration S_a for periods, $T \leq T_o$, is computed by the following equation. $S_a = PGA + \left[(S_{DS} - PGA) * \left(\frac{T}{T_o} \right) \right] \quad \text{Equation 12-45}$ <p>Where, S_{DS} is obtained in Step 1, T_o is obtained in Step 3, and PGA is obtained in Step 4.</p>
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. $S_a = \frac{S_{D1}}{T} \quad \text{Equation 12-46}$ <p>Where, S_{D1} is obtained in Step 2.</p>
8	The Site-Specific ADRS curve shall include the following items: <ul style="list-style-type: none"> • Site-Specific ADRS curve (both FEE and SEE as required) • Table of smoothed ADRS data values (T and S_a) • Provide the design spectral response parameters PGA, S_{DS}, S_{D1}; period markers T_o and T_s; M_w and R; PGV; D_{a5-95}; PGV; T'_o; T_0; $V^*_{s,H}$; H; and T_{NH}. An example of the information required is shown in Figure 12-11.



¹Source: Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7-10, ASCE, Reston, VA

Figure 12-10, Site-Specific Horizontal ADRS Curve Construction

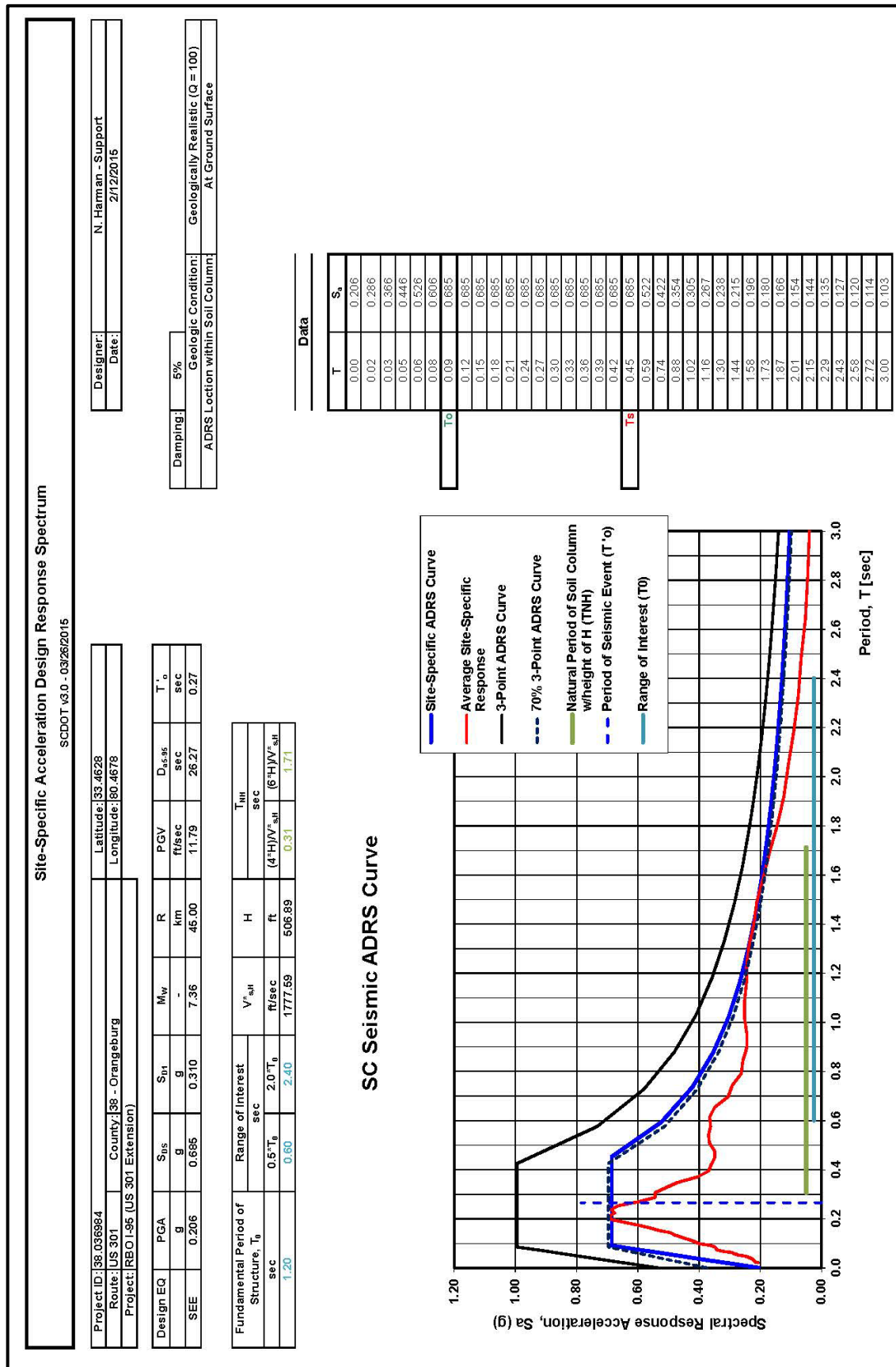


Figure 12-11, Site-Specific Horizontal ADRS Curve

12.10 GROUND MOTION DESIGN PARAMETERS

12.10.1 Peak Horizontal Ground Acceleration

The PGA at the ground surface is defined as the acceleration in the response spectrum obtained at a period, $T = 0.0$ seconds. If the 3-Point ADRS curves are used, the PGA obtained from Section 12.8.4 shall be used. If a SSRA is performed the spectral acceleration at period $T = 0.0$ second obtained from Site-Specific ADRS curve shall be used.

12.10.2 Earthquake Magnitude / Site-to-Source Distance

The M_W and R can be obtained from the seismic hazard deaggregations charts discussed in Chapter 11.

12.10.3 Seismic Event Predominant Period

The period of a seismic event should be determined in order to determine if the seismic input motion and the soils at a particular site match. If period matching occurs the potential for amplification of the ground motion at the site is possible. Matching of the period of the seismic event and the on-site soils may be termed harmonic resonance. The potential for significant damage may be magnified if the harmonic resonance includes not only the soil and seismic event having the same period but also the structure being designed. Therefore as indicated in Figures 12-8 and 12-11, the periods of the soil column, seismic event and structure should be indicated. The period of the soil column and seismic event are determined by the OES/GDS, with the period of the structure (first or fundamental period) by the SEOR. The period of the soil column is determined using the procedures provided in Section 12.3.3 and are based on actual site conditions.

The period of the seismic event is determined using the procedure provided by Rathje, Faraj, Russell and Bray (2004). In Rathje, et al. (2004) 4 different periods are discussed; T_m , T_{avg} , T_o and T_p . The development of T_m , the mean period, is discussed in Sections 12.8.2 and 12.8.3. T_{avg} , the average spectral period is not used. T_o is the smoothed spectral predominant period, is the period of the seismic event, while according to Rathje, et al. (2004) T_p , the predominant spectral period, should not be used. Therefore, the smoothed spectral predominant period, T_o , will be determined for each seismic event using the following equation. However, it should be noted that T_o is used in the development of the 3-Point ADRS curve as the beginning period of the flattened portion of the ADRS curve. Therefore, T'_o will be used to represent the smoothed spectral predominant period, not T_o as indicated in Rathje, et al. (2004).

$$T'_o = \frac{\sum \left[t * \ln \left(\frac{PSA_{B-C,t}}{PGA_{B-C}} \right) \right]}{\sum \ln \left(\frac{PSA_{B-C,t}}{PGA_{B-C}} \right)} \quad \text{Equation 12-47}$$

For spectral periods, t as defined in Table 12-6, where the $PSA_{B-C,t}$ meets the following criteria,

$$PSA_{B-C,t} = 1.2 * PGA_{B-C} \quad \text{Equation 12-48}$$

Substitute $PSA_{WR,t}$ and PGA_{WR} into Equations 12-47 and 12-48, if Weathered Rock conditions exist at the site as appropriate. The $PSA_{B-C,t}$, PGA_{B-C} , $PSA_{WR,t}$ and PGA_{WR} are determined from SCENARIO_PC.

Where,

t = Specific spectral period, second (see Table 12-6)

$PSA_{B-C,t}$ = Pseudo-acceleration at the B-C Boundary outcrop at a specific spectral period, from SCENARIO_PC (2006)

PGA_{B-C} = Pseudo Peak Ground Acceleration at the B-C Boundary outcrop at a spectral period of 0.0 seconds, from SCENARIO_PC (2006)

$PSA_{WR,t}$ = Pseudo-acceleration at the Weathered Rock (8,200 feet per second) outcrop at a specific spectral period, from SCENARIO_PC (2006)

PGA_{WR} = Pseudo Peak Ground Acceleration at the Weathered Rock ($V_s \geq 8,200$ feet/second) outcrop at a spectral period of 0.0 seconds, from SCENARIO_PC (2006)

12.10.4 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 M_w , R , and the earthquake significant duration as a function of acceleration (D_{a5-95}) can be computed by the following equation.

$R < 10$ km:

$$\ln(D_{a5-95}) = \ln \left\{ \frac{\left[\frac{\exp(5.204 + 0.851 * (M_w - 6))}{10^{(1.5 * M_w + 16.05)}} \right]^{-\left(\frac{1}{3}\right)}}{15.7 * 10^6} \right\} + 0.8664 \quad \text{Equation 12-49}$$

$R \geq 10$ km:

$$\ln(D_{a5-95}) = \ln \left\{ \frac{\left[\frac{\exp(5.204 + 0.851 * (M_w - 6))}{10^{(1.5 * M_w + 16.05)}} \right]^{-\left(\frac{1}{3}\right)}}{15.7 * 10^6} + 0.063 * (R - 10) \right\} + 0.8664 \quad \text{Equation 12-50}$$

Where:

M_w = Moment magnitude of design earthquake (FEE or SEE) Section 12.10.2

R = Site-to-source distance, kilometers, Section 12.10.2

D_{a5-95} = Seismic event significant duration, seconds

Kempton and Stewart (2006) developed a ground motion prediction equation to estimate the earthquake significant duration as a function of acceleration (D_{a5-95}) by using a modern database and a random-effects regression procedure. The correlation presented in the following equation uses the earthquake M_W , R , $V_{s,H}^* = V_{s,100}$, and depth-to-hard rock (d_{HR}) as defined in California to estimate the D_{a5-95} . Please note that Hard Rock in California is defined as having a $V_s \geq 5,000$ feet per second.

$$\ln(D_{a5-95}) = \ln \left\{ \frac{\left[\frac{\Psi}{\Upsilon} \right]^{-\frac{1}{3}}}{15.68 * 10^6} + K \right\} + \varepsilon \quad \text{Equation 12-51}$$

$$\Psi = \exp(2.79 + 0.82 * (M_W - 6)) \quad \text{Equation 12-52}$$

$$\Upsilon = 10^{(1.5 * M_W + 16.05)} \quad \text{Equation 12-53}$$

$$K = 0.15 * R + 3.00 - 0.0041 * \left(\frac{V_{s,100}^*}{3.2808} \right) + 0.0012 * d_{HR} \quad \text{Equation 12-54}$$

Where:

$V_{s,100}^*$ = Site stiffness with $Z_{DTM}=0$, ft/sec (Section 12.3.2)

M_W = Moment magnitude of design earthquake (FEE or SEE) Section 12.9.2

R = Site-to-source distance, kilometers, Section 12.10.2

d_{HR} = Depth from ground surface to hard rock ($V_s \geq 5,000$ ft/sec (1,500 m/s)), m

ε = Near-fault forward directivity correction for earthquakes (dip-slip or strike-slip faults)

R < 20 km:

$$\varepsilon = 0.015 * (R - 20) \quad \text{Equation 12-55}$$

R ≥ 20 km:

$$\varepsilon = 0 \quad \text{Equation 12-56}$$

The Kempton and Stewart (2006) study confirmed the previous correlations (i.e., Abrahamson and Silva (1996)) that D_{a5-95} increased with an increase in M_W and R . In addition, the study found that the D_{a5-95} significantly increased with decreasing $V_{s,H}^*$. The D also increased slightly with an increase of depth-to-hard rock (d_{HR}).

South Carolina shear wave profiles have indicated that site stiffness $V_{s,H}^*$ can vary significantly across the state from greater than 5,000 ft/s (1,500 m/s) to less than 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 12-12. An $M_W = 7.3$ and a $d_{HR} = 2,600$ feet (800m) have been selected as typical of the lower South Carolina Coastal Plain. The Abrahamson and Silva (1996) relationship for rock has also been plotted for reference.

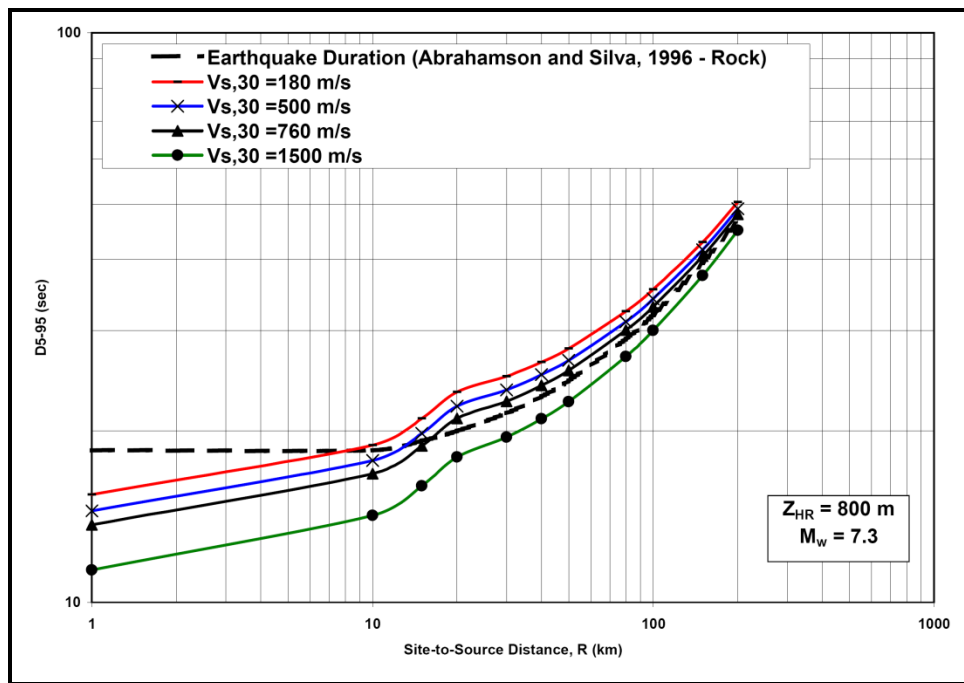


Figure 12-12, Effects of Site Stiffness on Earthquake Duration

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 12-13. The Abrahamson and Silva (1996) relationship for rock has also been plotted as a reference.

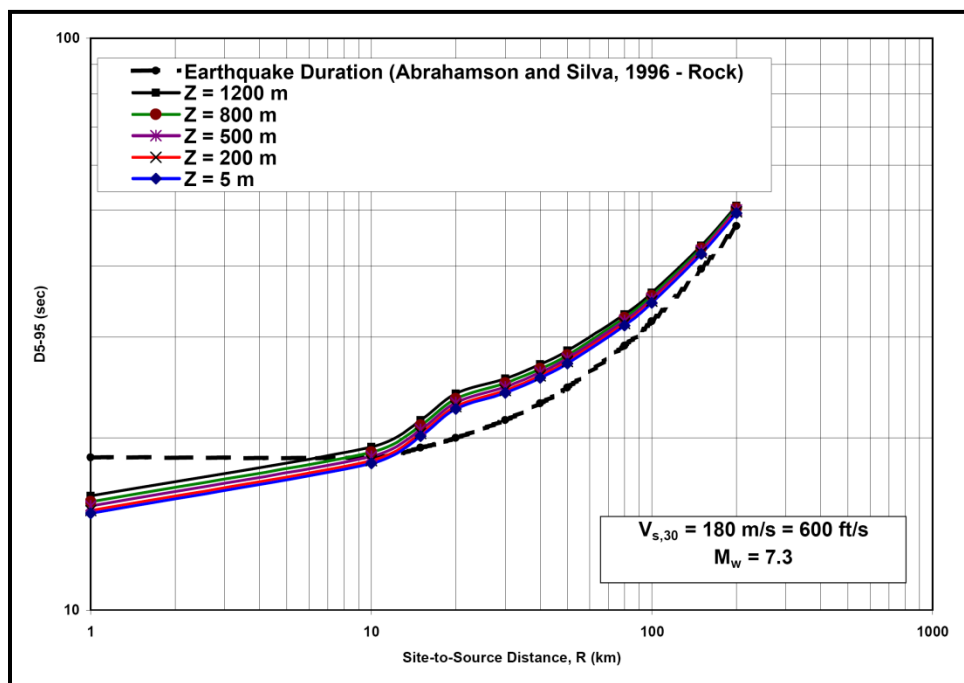


Figure 12-13, Effects of Depth-to-Hard Rock on Earthquake Duration

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

12.10.5 Energy Content

According to Kavazanjian, et al. (2012),

The *energy content* of the acceleration time history provides another means of characterizing quantitatively the intensity of strong ground motions. The energy content of a strong ground motion record is proportional to the square of the acceleration. In engineering practice, the energy content of the motion is typically expressed in terms of either the *root-mean-square* and *duration* of the acceleration time history or the *Arias Intensity*, I_A . The *Arias Intensity*, I_A , is proportional to the square of the acceleration integrated over the entire acceleration time history:

$$I_A = \frac{\pi}{2g} \int_0^{t_f} [a(t)]^2 dt \quad \text{Equation 12-57}$$

where $a(t)$ is the time history of acceleration (the accelerogram), g is the acceleration of gravity and t_f is the duration of the shaking. Arias (1969) showed that this integral is a measure of the total energy of the accelerogram.

The root-mean-square of the acceleration time history, or *RMSA*, is the square root of the square of the acceleration integrated over the duration of the motion and divided by the duration:

$$RMSA = \sqrt{\frac{1}{t_f} \left\{ \int_0^{t_f} [a(t)]^2 dt \right\}} \quad \text{Equation 12-58}$$

where $a(t)$ is the acceleration time history, and t_f is the duration of the strong ground shaking. The RMSA represents an average value of acceleration over the duration of strong shaking. The square of RMSA multiplied by the duration of the motion is directly proportional to the energy content of the motion, i.e., Arias intensity is related to the RMSA as follows:

$$I_A = \frac{\pi}{2g} (RMSA)^2 * t_f \quad \text{Equation 12-59}$$

The value of the Arias Intensity is independent of the duration of strong shaking, while RMSA depends upon the definition of the strong shaking duration. However, as the energy content of the motion is fixed, the product of the RMSA and the squared duration will remain constant as suggested in *Equation 12-59*. The definition of the duration of strong shaking for an acceleration time history can be somewhat arbitrary, as relatively low intensity motions may persist for a long time towards the end of a strong motion record. If the defined duration of strong motion is increased to include these low intensity motions, the Arias Intensity will remain essentially constant but the RMSA will decrease. Therefore, some investigators prefer Arias Intensity to RMSA as a measure of energy content, as the Arias Intensity is essentially a fixed value while the RMSA depends upon the definition of the duration of strong ground motion.

Arias Intensity and/or RMSA and duration are useful parameters in selecting time histories for geotechnical analysis. This is particularly true if a seismic deformation analysis is to be performed, as the deformation potential of a strong motion records is related to the energy content, which can be expressed as a function of either Arias Intensity or the product of the RMSA and duration of the records.

The duration of shaking (t_f) discussed above may be taken as D_{a5-95} as discussed in Section 12.10.4. The use of D_{a5-95} as the duration of shaking is only an approximation; the actual t_f should be obtained from a time series.

12.10.6 Peak Ground Velocity

The peak ground velocity, PGV, of the earthquake can be determined from a site-specific response analysis. If the 3-Point ADRS curves are developed, PGV correlations based on the Anderson, Martin, Lam, and Wang (2008) may be used.

The mean PGV, in units of in/sec can be computed by the following equation.

$$PGV = V_{Peak} = 38 * (F_v * S_1) = 38 * S_{D1} \quad \text{Equation 12-60}$$

Anderson, et al. (2008) recommends using the mean plus one standard deviation value for determining the PGV, to provide a margin of conservatism, using the following equation.

$$PGV = V_{Peak} = 55 * (F_v * S_1) = 55 * S_{D1} \quad \text{Equation 12-61}$$

Where,

F_v = Site coefficient defined in Sections 12.8.2 and 12.8.3, 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 12.8 and 11.8.2 at the B-C Boundary or Hard Rock outcrop

S_{D1} = Design long-period (1.0 second = 1 Hz) spectral response acceleration parameter

However, Kavazanjian, et al. (2012) recommends the use of Equation 12-60, which is more consistent with LRFD principles, than Equation 12-61.

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