Table 2.0-1R (Sheet 10 of 14)
Key Site Parameters

CP COL 2.3(1)

CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)

CP COL 2.5(1)

Geology, Seismology, and Geotechnical Engineering		
Parameter Description	**Parameter Value**	
	DCD	**CPNPP 3 and 4**
Maximum slope for foundation-bearing stratum	20° from horizontal in untruncated strata	Layer C slopes at less than 10 degree across the footprint and the site area.
Safe-shutdown earthquake (SSE) ground motion	0.3 g peak ground acceleration	The SSE is the envelope of the GMRS and the minimum earthquake requirement of 10 CFR 50 Appendix S, based on the shape of the Certified Site Design Response Spectra (CSDRS) scaled down to a PGA of 0.1 g. The CSDRS is itself a modified RG 1.60 shape formed by shifting the control points at 9 Hz and 33 Hz to 12 Hz and 50 Hz, respectively.
SSE (certified seismic design) horizontal ground response spectra	Regulatory Guide (RG) 1.60, enhanced spectra in high frequency range (see Figure 3.7.1-1)	The minimum DCD spectrum envelops all four FIRS, down to frequencies of 0.5 Hz. ~~For this reason, detailed spectral shapes were not fit to the FIRS spectra between the seven spectral frequencies for which ground motion equations are available.~~ Values of the horizontal 10^{-5} UHRS and FIRS are shown in Table 2.5.2-229 for the seven spectral frequencies.

RCOL2_03.08.04-72

Table 2.0-1R (Sheet 11 of 14)
Key Site Parameters

CP COL 2.1(1)
CP COL 2.2(1)
CP COL 2.3(1)
CP COL 2.3(2)
CP COL 2.3(3)
CP COL 2.4(1)
CP COL 2.5(1)

SSE (certified seismic design) vertical ground response spectra	RG 1.60, enhanced spectra in high frequency range (see Figure 3.7.1-2)	For vertical FIRS motions, the same considerations used for the GMRS were used for the FIRS. That is, as a conservative assumption the V/H ratio for the FIRS spectra is assumed to be equal to the V/H ratio from RG 1.60.~~That is, for large source-to-site distances, results in the US APWR DCD indicate that V/H ratios will be less than unity for all frequencies. V/H ratios are likely to be considerably less than unity at frequencies below 5 Hz. Appendix J of the DCD indicates that for distances exceeding 40 km, soil sites in both the WUS and CEUS will have V/H ratios of 0.5 or less. Thus it is reasonable to assume that vertical FIRS will be enveloped by the vertical minimum DCD spectrum.~~	RCOL2_03.08.04-72 CTS-00916 CTS-00916
Potential for surface tectonic deformation at site	None within the exclusion area boundary	No potential tectonic surface deformation has been identified at the site.	
Subsurface stability – ~~average~~minimum allowable static bearing capacity	15,000 lb/ft^2	The ~~average~~minimum allowable bearing capacity of the foundation bearing stratum meets or exceeds the DCD requirement	CTS-01125
Subsurface stability – ~~average~~minimum allowable dynamic bearing capacity, normal conditions plus SSE	~~95,000~~60,000 lb/ft^2	The ~~average~~minimum allowable dynamic bearing capacity of the foundation bearing stratum meets or exceeds the DCD requirement	
Subsurface stability – minimum shear wave velocity at SSE input at ground surface	1000 ft/s	The site stratigraphy has a measured velocity in excess of 1000 ft/sec	
Subsurface stability – shear wave velocity for defining firm rock	3500 ft/s	The site meets the minimum 3500ft/sec for a firm rock site	
Subsurface stability – shear wave velocity for defining firm to hard rock	6500 ft/s	The site does not meet the Vs for a firm to hard rock site	

232). Generally, all publicly accessible locations in and around the site area were visited in order to verify the accuracy of the site area map, to search for signs of deformation (faulting or folding) in bedrock and surficial outcrops, and to search for paleoliquefaction features. Minor flexures, limited in both vertical and lateral extent (less than 3 ft and 40 ft, respectively) have been noted in surrounding exposures of the Glen Rose Formation. However the limited extent and lack of evidence of offset or brittle deformation indicate that these flexures may be related to non-tectonic factors such as differential consolidation, or dissolution of underlying sediments. This interpretation is strengthened by the observation that underlying beds do not mimic the structure in the case of the fold in the Glen Rose Formation. A review of the core that was obtained from the borings drilled as part of the site geotechnical investigation (discussed in Subsection 2.5.4) shows no evidence for brittle or ductile deformation that can be related to seismic activity.

2.5.1.2.5.2 Zones of Weathering, Alteration or Structural Weakness

The area for CPNPP Units 3 and 4 is cut to a yard grade of elevation 822 ft msl and all weathered materials are to be removed, as discussed in Subsection 2.5.4.5.1. All Category 1 structures ~~(with the exception of shallow embedded electrical duct banks)~~ are founded directly on a thick (average 65 ft), laterally extensive, limestone unit within the Glen Rose Formation, at about elevation 782 ft msl. Subsection 2.5.4 discusses these conditions, including the static and dynamic properties of this and other subsurface layers. Site reconnaissance of exposures surrounding the site, a review of aerial photography, and examination of borings drilled as part of the geotechnical investigation showed no zones of enhanced weathering or structural weakness such as fractures or joints. Also, petrographic analysis of samples acquired from core borings drilled as part of the geotechnical investigation, as well as samples taken from exposures surrounding the site, does not indicate any secondary alteration of minerals. RCOL2_03.08.04-63

Reconnaissance of the site area included several of the incised ephemeral drainage valleys and outcrops of the Glen Rose Formation. As described in Subsection 2.5.1.2.3, with the exception of limited Paluxy Formation exposures, most of the outcroppings at the surface, and within incised valleys surrounding the CPNPP site, consist of shales and limestones of the Glen Rose Formation. The surface of the Glen Rose is typically a weathered mantle ranging from 0 to several ft thick, as noted from reconnaissance studies of the site area as well as from borings drilled as part of the geotechnical investigation discussed in Subsection 2.5.4. It was also noted from the geotechnical borings that the weathered zone, discounting areas where fill has been placed, ranges from 0 to about 10 ft thick and terminates abruptly at competent rock with rock quality designations (RQD) and percent recoveries of core of greater than 90 percent. It is also noted from the geotechnical borings that bedding planes within the shale layers are primarily horizontal and develop partings along these planes as the material desiccates. These shale layers overlie the limestone foundation-bearing unit and will be excavated and removed during construction. Excavation walls will require engineered reinforcement as these shale layers exist near the base of the excavation as described in Subsection 2.5.4.

Hz, 2.5 Hz, 1 Hz, and 0.5 Hz). Anchoring the LF spectral shape to all frequencies was necessary because otherwise the LF spectral shape exceeded the HF spectral shape at high frequencies. The use of a LF shape with amplitudes higher than the HF UHRS amplitudes would not be appropriate because this would overdrive the soil column. Anchoring the LF spectrum to the UHRS amplitudes at all frequencies ensures that appropriate ground motions are represented. The lack of fit of the LF spectral shape to the HF UHRS amplitudes results from distant, large earthquakes that contribute to seismic hazard at this site, with ground motion ε values greater than unity. In these cases, the spectral shapes of NUREG/CR-6728 are not appropriate and the LF spectrum needs to be anchored to the HF UHRS amplitudes.

Figures 2.5.2-229 through 2.5.2-231 show the smooth horizontal HF and LF UHRS calculated in this way for 10^{-4}, 10^{-5}, and 10^{-6} annual frequencies of exceedance, respectively. As mentioned previously, these spectra accurately reflect the UHRS amplitudes in Table 2.5.2-219 that were calculated for the seven spectral frequencies at which PSHA calculations were done. Because the HF and LF spectra were scaled to the same high-frequency amplitudes, they are very similar at high frequencies and differ only for frequencies below 5 Hz. As a result of these similarities, a broad-banded spectrum was used as input to site response calculations, using the envelope of the HF and LF spectra shown in Figures 2.5.2-229 through 2.5.2-231.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.5 with the following.

The subsurface conditions necessary to predict and model the seismic wave transmission characteristics for CPNPP Units 3 and 4 were determined from both site-~~-~~specific and regional data. ~~This~~These data included both stratigraphic and representative shear and compressional wave measurements that were used to develop the site profile and ~~is~~are summarized in Table 2.5.2-227. A detailed discussion of the data and methodology for developing the stratigraphy and corresponding dynamic properties used to define the dynamic profile for the site is provided in Subsection 2.5.4.4.2.2. CTS-00916 RCOL2_02.05.02-16 S01 RCOL2_02.05.02-16 S01 CTS-00916

The profile is divided into the shallow profile (surface to about 500 ft) and the deep profile (about 500 ft to “basement”). The shallow profile represents depth to which extensive characterization has been performed. The lateral and vertical control on the subsurface strata (layering) was defined primarily on lithology and material properties. The velocity measurements in the shallow profile have been developed from 15 suspension logs from borings drilled as part of the foundation exploration described in Subsection 2.5.4.4.2.1.

The foundation basemats of all ~~category 1~~seismic Catergory I structures will be founded on a limestone unit (denoted as Layer C in Subsection 2.5.4)~~, with the exception of category 1 electrical duct banks that will be embedded in compacted fill adjacent to the nuclear island.~~ Excavation to Layer C will remove the shallower CTS-00916 RCOL2_03.08.04-63 CTS-00916

than 10^{-4}. This means that the frequency of damaging ground motions ~~at the GMRS elevation~~ is less than 10^{-4}. Under these conditions, the GMRS is calculated from Equation 7 above as 0.45 x SA(10^{-5}). Table 2.5.2-228 shows the 10^{-5} ground motion at the seven spectral frequencies for which ground motion equations are available, and shows the GMRS calculated as 0.45 x SA(10^{-5}). RCOL2_03.07.02-1

Figure 2.5.2-234 shows the horizontal GMRS spectrum taken from Table 2.5.2-228, plotted with the horizontal DCD spectrum. This figure shows that the GMRS at the seven spectral frequencies at which ground motion equations were available from the 2004 EPRI study (Reference 2.5-401) is enveloped by the DCD spectrum.~~Figure 2.5.2-234 shows the horizontal GMRS spectrum taken from Table 2.5.2-228, plotted with the horizontal CSDRS. This shows that the GMRS down to 0.5 Hz is enveloped by the CSDRS. As a result, extensive fitting of spectral shapes between the seven spectral frequencies indicated in Table 2.5.2-213 is not undertaken.~~ RCOL2_03.08.04-72 RCOL2_03.07.02-1

~~A seismic hazard calculation was made using the site amplification factors for the GMRS and four FIRS conditions (FIRS2, FIRS3, FIRS4, and FIRS4-CoV50). These calculations were made at the seven spectral frequencies at which ground motion equations were available from the EPRI (2004) study (100 Hz, 25 Hz, 10 Hz, 5Hz, 2.5 Hz, 1Hz,and 0.5 Hz). The CAV filter was applied for these calculations, and at all spectral frequencies, the 1E-4 amplitudes were zero (i.e. the highest hazard at low amplitudes was less than 1E-4). As a result, the GMRS and FIRS amplitudes were determined from (for example) GMRS = 0.45 x SA(l0^{-5}) where SA(l0^{-5}) is the spectral acceleration for l0^{-5} annual frequency of exceedence.~~

The horizontal ~~1E-5~~10^{-5} and GMRS spectra were calculated at 39 frequencies between 0.1 Hz and 100 Hz for the GMRS elevation. This spectral frequency range encompasses all the energy of the rock ground motions for earthquakes in the Central and Eastern United States and meets the requirements in Subsection 3.4 “Hazard Assessment” in item C “Regulatory Position” of Regulatory Guide 1.208. The natural frequency of the GMRS soil column is 0.29 Hz. Because of the very flat appearance of the spectra at the seven spectral frequencies at which hazard calculations were made, log-log interpolation between available hazard values was used, with the exception of the following frequency ranges. CTS-00916 RCOL2_03.07.02-1

1 Hz to 5Hz: Within this frequeney range, a peak ~~inside~~in site spectra occurs at 2.5 Hz, reflecting a site amplification at about 2 Hz. To reflect this amplification, the ~~1E-5~~10^{-5} spectral amplitude at 2.5 Hz was broadened using rock spectral shapes from NUREG/CR-6728 and using the broad-banded values of M=~~7.7~~7.5 and R=~~890~~650 km for ~~1E-5~~10^{-5} (on which the site amplification calculations were based). This is an acceptable approximation given that the rock spectrum is decreasing between 2.5 and 1 Hz. RCOL2_02.05.02-16 S01 CTS-00916 RCOL2_03.07.02-1

0.5 Hz to 0.1 Hz: Below 0.5 Hz, ~~the assumption was made that spectral accelerations are proportional to *f* down to 0.125 Hz (where *f* is frequency), and are proportional to f^2 between 0.125 Hz and 0.1 Hz. This is a common assumption for spectral shapes at low frequencies.~~the site-specific spectral shape determined during site amplification calculations was used to extrapolate to 0.1 Hz. This spectral shape was determined from the 10^{-5} surface spectrum at the GMRS elevation, using the 10^{-5} rock input motion. This spectral shape between 0.5 Hz and 0.1 Hz was used to extrapolate the GMRS from 0.5 Hz to 0.1 Hz. The GMRS shape at long periods is thereby consistent with the site-specific amplification calculation for the GMRS elevation. CTS-01098 RCOL2_03.07.02-1 RCOL2_02.05.02-16 S01 RCOL2_03.08.04-72

~~Spectra for the four FIRS conditions (FIRS2, FIRS3, FIRS4, and FIRS4-CoV50 were calculated in a similar way. Note that the FIRS3 spectra have peaks at about 2 Hz and 10 Hz, and that the FIRS4 and FIRS4-CoV50 spectra have peaks at about 1.5 Hz and 5 Hz. These peaks were broadened in an approximate way similar to the procedure used for the GMRS.~~ RCOL2_02.05.02-16 S01

~~These GMRS and FIRS spectra are plotted in Figures 2.5.2-247 through 2.5.2-251 with the 1E-5 spectrum for each condition also plotted. Table 2.5.2-236 shows the numerical values for the 1E-5 and GMRS spectra, and Table 2.5.2-237 shows the numerical values for the 1E-5 and FIRS spectra.~~The horizontal GMRS and 10^{-5} spectra are plotted in Figure 2.5.2-247, and the numerical values of the spectra are shown in Table 2.5.2-236. RCOL2_03.08.04-72

The smooth horizontal GMRS spectrum is plotted in Figure 2.5.2-257 along with the respective DCD spectrum. This figure shows that the GMRS spectrum is enveloped by the DCD.

2.5.2.6.1.2 Vertical GMRS Spectrum

Vertical motions at the CPNPP Units 3 and 4 site are addressed by reviewing results in NUREG/CR-6728 for V/H ratios at deep soil sites, for both the western US (WUS) and the CEUS. Example results presented in the ~~US APWR DCD~~NUREG/CR-6728 indicate that for earthquakes >40 km from a deep soil site, V/H ratios are expected to be less than unity for all frequencies (Figures J-31 and J-32 in Appendix J of the ~~DCD~~NUREG/CR-6728). For the 10^{-5} ground motion, expected distances from deaggregation are greater than 100 km (Table 2.5.2-220). Any exceedance of unity occurs for high frequencies (>10 Hz) for short source-to-site distances. Also, for ground motions with peak horizontal accelerations <0.2g, the recommended V/H ratios for hard rock conditions are less than unity; see Table 4-5 of the ~~DCD~~NUREG/CR-6728. The conclusion is that V/H ratios for the CPNPP Units 3 and 4 site will be less than unity for all spectral frequencies. Therefore, the vertical GMRS will be below the horizontal GMRS shown in Figure 2.5.2-~~233~~234. CTS-00916 CTS-00916 CTS-00916 RCOL2_02.05.02-16 S01

Figure 2.5.2-234 shows that the horizontal DCD spectrum exceeds the horizontal GMRS. The vertical DCD spectrum equals or does not exceed the horizontal DCD

spectrum for frequencies above 3.5 Hz. The conclusion is that the vertical DCD spectrum will also exceed the vertical GMRS. Under this condition, the DCD minimum vertical design motion will govern the vertical response, just as the DCD minimum horizontal design motion will govern the horizontal response.

Vertical GMRS and FIRS spectra were developed using vertical-to-horizontal (V/H) ratios. NUREG/CR-6728 and RG 1.60 indicate proposed V/H ratios for design spectra for nuclear facilities, and these V/H ratios are plotted in Figure 2.5.2-252. The V/H ratios in Figure 2.5.2-252 taken from NUGREG/CR-6728 (the blue curve) are recommended for hard sites in the CEUS. The Comanche Peak site is a deep, soft-rock site with shale~~s~~ and limestone~~s~~ near the surface having shear-wave velocities of about 2600 fps, and the V/H ratios for this site condition will be similar to those for hard roick sites. CTS-00916

Based on these comparisons, it is concluded that the applicable V/H ratios at the Comanche Peak site will be ≤ 1.0 at all spectral frequencies between 100 Hz and 0.1 Hz. As a conservative assumption, the V /H ratio is assumed to be equal to ~~1.0 at all spectral frequencies~~the V/H ratio from RG 1.60. This assumption is also plotted in Figure 2.5.2-252. The vertical GMRS spectrum resulting from this assumption is presented in Table 2.5.2-236. RCOL2_03.08.04-72

~~The result of this assumption is that the spectra plotted in Figures 2.5.2-247 through 2.5.2-251 for the GMRS and four FIRS conditions apply to both the horizontal and vertical motions.~~

~~Tables 2.5.2-236 and 2.5.2-237 document (respectively) the 10^{-5} UHRS and GMRS, and the 10^{-5} UHRS and FIRS. Because V /H is assumed to be equal to unity, these spectra apply to both horizontal and vertical motions.~~The smooth vertical GMRS spectrum is plotted in Figure 2.5.2-258 along with the respective DCD spectrum. This figure shows that the GMRS spectrum is enveloped by the DCD.

2.5.2.6.2 Foundation Input Response Spectrum

Site response analyses were conducted for an additional four cases (FIRS 2, FIRS 3, FIRS 4_CoV30, and FIRS 4_CoV50) to consider foundation input response spectra for specific conditions different from the GMRS elevation. These four cases are as follows:

FIRS 2 - Set at elevation 787 ft.

This FIRS represents generic site response conditions for structures resting on fill concrete layer in which the fill concrete thickness and horizontal extent away from the edge of the foundation is significant and thus modeled as a horizontally infinite layer.

- FIRS 2 analysis demonstrates that the response at the top of the fill concrete remains well below the minimum earthquake and does not apply to any specific structure.

The FIRS 2 profile consists of 5 ft of fill concrete placed over a sub-excavated stiff limestone (Layer C) surface at elevation 782 ft. Fill concrete with compressive strength ranging from 2,500 psi to 4,400 psi is considered by using a mean shear wave velocity of 6800 fps with a range of +/- 500 fps. See Table 2.5.2-227 for properties used for FIRS 2 analysis. Note that the site-specific soil-structure interaction analyses described in Subsection 3.7.2 model the fill concrete under the category 1 foundations as part of the structural model.

FIRS 3 - Set at Plant Grade elevation 822 ft.

The FIRS 3 profile considers the ground surface seismic response in areas of the site where cutting of the native soil is required to reach final Plant Grade elevation 822 ft.

- FIRS 3 analysis demonstrates that the response at Plant Grade elevation in regions of the site with native soil remains below the minimum earthquake. It does not represent the foundation subgrade elevation for any safety-related facilities identified, but could accommodate possible future shallow (at-grade) facilities.

The profile consists of stiff limestone at elevation 782 ft and overlying shale (Glen Rose Layer B1 and B2) and interbedded limestone/shale (Glen Rose Layer A) to Plant Grade elevation 822 ft. See Table 2.5.2-227 for properties used for FIRS 2 analysis.

FIRS 4 - Set at Plant Grade elevation 822 ft:

- FIRS 4 analysis demonstrates that the response of engineered compacted backfill at Plant Grade elevation remains below the minimum earthquake. ~~This FIRS corresponds to Emergency Power electric cable conduit duct banks that are shallowly embedded.~~ RCOL2_03.08.04-63

The elevations of FIRS 4 and FIRS 3 are identical, but this profile consists of sub-excavation to stiff limestone at elevation 782 ft, and backfilling to Plant Grade with cohesionless engineered fill to Plant Grade elevation 822 ft. Assumed shear wave velocity and shear modulus/damping properties for the fill are estimated based on a specified range of cohesionless fill materials, and reported properties for similar compacted fill materials. Ranges of values representing best estimates, and lower and upper bounding values, are provided in Table 2.5.2-227. Degradation curves are provided in Figure 2.5.2-232. FIRS 4 consists of two different cases (FIRS~~-~~4_CoV30 and FIRS4_CoV50) to provide a wide variability on shear wave velocities estimated for the cohesionless compacted fill. CTS-01098

FIRS4_CoV30: elevation 822 ft. The elevation for FIRS 4 is the same as for FIRS 3, but the profile consists of sub-excavation to stiff limestone at elevation 782 ft, and backfilling to plant grade with cohesionless engineered compacted fill.

FIRS4_CoV50: elevation 822 ft. This profile is the same as for FIRS 4 except it uses a coefficient of variation (CoV) of 50% (instead of 30%) for the Vs of the fill material.

Figures 2.5.2-235 through 2.5.2-238 show median amplification factors and logarithmic standard deviations for these four FIRS cases, for the 10^{-4}, 10^{-5}, and 10^{-6} broadband input motions.

The seismic hazard for each FIRS case was calculated by integrating the horizontal amplification factors shown in Figures 2.5.2-235 through 2.5.2-238 with the rock hazard and applying the CAV filter. This is an analogous calculation to the calculation of hazard for the GMRS elevation. For all FIRS cases the hazard curves at low amplitudes rolled over to an annual frequency of exceedance that was less than 10^{-4}. As was the case for the GMRS, the FIRS spectra were calculated using the 10^{-5} UHRS and applying the factor from Eq. 2.5.2-3; i.e., FIRS = 0.45 × SA(10^{-5}).

Figure 2.5.2-239 plots the four horizontal FIRS and compares them to the horizontal minimum DCD spectrum. The minimum DCD spectrum envelops all four FIRS, down to frequencies of 0.5 Hz. ~~For this reason, detailed spectral shapes were not fit to the FIRS spectra between the seven spectral frequencies for which ground motion equations are available.~~ Values of the horizontal 10^{-5} UHRS and FIRS are shown in Table 2.5.2-229 for the seven spectral frequencies. RCOL2_02.05.02-16 S01 RCOL2_02.05.02-16 S01

Smooth horizontal spectra for the four FIRS conditions (FIRS2, FIRS3, FIRS4, and FIRS4-CoV50) were calculated in a manner similar to the way in which the smooth GMRS was calculated, as described in Section 2.5.2.6.1.1. Note that the FIRS3 spectra have peaks at about 2.5 Hz and 10 Hz, and that the FIRS4 and FIRS4-CoV50 spectra have peaks at about 1.5 Hz and 5 Hz. These peaks were broadened in an approximate way similar to the procedure used for the GMRS. RCOL2_03.07.02-1 RCOL2_03.08.04-72

The horizontal 10^{-5} and FIRS spectra are plotted in Figures 2.5.2-248 through 2.5.2-251. Table 2.5.2-237 shows the numerical values for the 10^{-5} and FIRS spectra.

For vertical FIRS motions, the same considerations used for the GMRS were used for the FIRS. ~~That is, for large source-to-site distances, results in the US APWR DCD (Reference 2.5.2-288) indicate that V/H ratios will be less than unity for all frequencies. V/H ratios are likely to be considerably less than unity at frequencies below 5 Hz. Appendix J of Ref 2.5.2-288 indicates that for distances exceeding 40 km, soil sites in both the WUS and CEUS will have V/H ratios of 0.5 or less. Thus, it is reasonable to assume that vertical FIRS will be enveloped by~~ CTS-00916 RCOL2_03.08.04-72

~~the vertical minimum DCD spectrum.~~That is, as a conservative assumption the V/H ratio for the FIRS spectra is assumed to be equal to the V/H ratio from RG 1.60. RCOL2_03.08.04-72

The smooth horizontal and vertical FIRS spectra are plotted in Figures 2.5.2-257 and 2.5.2-258, respectively, along with the respective DCD spectrum. These figures show that the FIRS spectra are enveloped by the DCD.

Of particular note is engineering Layer C, which is a massive limestone layer within the Glen Rose Formation. This limestone layer provides the foundation bearing layer for all seismic category I and II structures ~~(with the exception of shallow embedded electrical duct banks)~~. The following discussions will refer to these engineering layers and respective geologic formations interchangeably as appropriate. A detailed discussion of the engineering stratigraphy is provided in Subsection 2.5.4.3.1. RCOL2_03.08.04-63

2.5.4.1 Geologic Features

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.1 with the following.

This subsection discusses site-specific geologic processes, materials, and conditions relative to the potential for foundation instability or adverse performance at the CPNPP Units 3 and 4 site. Subsections 2.5.1 and 2.5.3 provide background discussion regarding the geologic setting and regional framework, tectonic setting and history, and potential geologic hazards that are referenced or expounded upon in this section. The focus of the following discussion is a site-specific evaluation of these geologic conditions and features within the CPNPP Units 3 and 4 plant foundation and influence zone pertaining to the following issues:

- Geologic Stress Conditions and Structure
- Adverse Mineralogy and Zones of Weathering or Alteration
- Karst or Zones of Dissolution
- Tectonic Ground Failure (Paleoearthquake)
- Landslides and Slope Stability
- Non-Tectonic Deformation and Volcanism
- Groundwater Conditions/Withdrawal
- Man-Induced Activity

2.5.4.1.1 Geologic Stress Conditions and Structure

The CPNPP site is located within a stable continent area (Subsection 2.5.3) with relatively low stress conditions and no active structural deformation within the 25 mi radius around the site. Late Cretaceous bedrock of the Glen Rose and Twin Mountain formations that underlie the site and locally crop out at the surface is nearly horizontal bedded and undeformed. Individual rock strata have been confidently identified and traced through the plant site, and extend to correlative elevations in boring logs and construction excavation photographs from the CPNPP Units 1 and 2, located about 2000 ft to the east. Figures 2.5.4-204 and

2.5.4.2.3.4.4 Organic Content

Organic contents were measured in two samples of fine-grained residual soils. Results indicate an organic content of 1.9 percent for a sample of sandy clay and 2.6 percent for a sample of silty clay.

2.5.4.3 Foundation Interfaces

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.3 with the following.

The following subsections describe the subsurface conditions determined from the extensive investigation and resulting data. The boring data, including detailed core descriptions, geophysical logs and surveys and laboratory test results, are used to divide the vertical section into layers that are distinguished by different physical characteristics. These engineering layers were applied to develop a representative static and dynamic profile for engineering analysis as well as development of the seismic ground motion for the site, as described in Subsection 2.5.2. Significant discussion is focused on a prominent and thick limestone layer (referred to as engineering Layer C), the top of which is present at about 40 ft below the yard grade (elevation 822 ft). This limestone layer is the foundation bearing layer for all seismic category I structures~~, with the exception of shallow embedded electrical duct banks~~. It has a uniform thickness of about 60 ft and a consistent S-wave velocity of about 6300 fpc. Subsurface conditions to a depth of about 550 ft are described in the following subsections. RCOL2_03.08.04-63

2.5.4.3.1 Engineering Stratigraphy

The subsurface conditions and engineering stratigraphy for the site area are based on the integrated data acquired from the geotechnical exploration program described in Subsection 2.5.4.2 and shown on Figure 2.5.4-202. Figures 2.5.4-206, 2.5.4-207, and 2.5.4-208 are examples of boring in situ test summary logs from key boreholes that integrate geologic and geophysical data to help define and correlate engineering layers through the site.

Site bedrock materials are divided into discrete engineering layers for evaluation of foundation and seismic site response characteristics. The bedrock formations extending from the ground surface to a depth of about 550 ft (approximately elevation 294 ft) are divided into 13 stratigraphic-engineering (engineering) rock layers (Figures 2.5.4-204 and 2.5.4-205), and a thin cover of surface residual soils and localized undocumented fill. Engineering rock layers are correlated with the regional geologic stratigraphy described in Subsection 2.5.1, and rock strata defined for the CPNPP Units 1 and 2 FSAR that include the Glen Rose Formation, Twin Mountain Formation, and Mineral Wells Formation. Figure 2.5.4-205 shows the correlation between the site engineering layers and those defined for CPNPP Units 1 and 2. Each engineering layer is a unique stratigraphic layer differentiated on the basis of lithology (e.g., shale or limestone), rock mass property (e.g., degree of fracturing or cementation), geotechnical index properties (e.g., plasticity, shear strength), and geophysical characteristics (e.g., seismic wave

2.5.4.4.2.1 Shallow Seismic Profile

The shallow velocity profile (extending to the maximum depth of site exploration) has been extensively characterized by 141 geotechnical borings, 20 monitoring well stratigraphic profile borings, and site geologic mapping. These data were used to develop the site engineering stratigraphy that was demonstrated to closely match the velocity layering in the 15 P-S Suspension borings.

The foundation basemats of all seismic category I and II structures will be founded on a limestone layer (engineering Layer C)~~, with the exception of seismic category I duct banks that will be embedded in compacted fill adjacent to the nuclear island~~. Excavation to Layer C will remove the shallower materials (Layers A, B1 and B2). Where the top of Layer C is below the bottom of the foundation elevation, fill concrete will be placed to achieve the bottom of basemat elevation. The average thickness of Layer C is greater than 60 ft and dips less than 1 degree. The average S-wave velocity of Layer C is greater than 6000 fps. Profiles for development of the GMRS and FIRS are detailed in Subsection 2.5.2. RCOL2_03.08.04-63

Borehole-to-borehole correlation of engineering layers and respective P-S Suspension-log velocity measurements were used to develop velocity profiles for each of the fifteen suspension-log borings as well as a representative integrated site profile to a depth of about 500 ft. The mean engineering layer surface elevations and thicknesses were used to produce the integrated shallow seismic velocity profile. The mean layer boundaries determined from the entire set of geotechnical core borings and monitoring well profile holes described in Subsection 2.5.4.2 were used to evaluate potential lithologic variability and geologic influence on seismic wave transmission properties.

A summary plot of measured P- and S-wave velocity versus elevation is included on Figure 2.5.4-239. Spatial distribution of velocities for each engineering layer was evaluated to determine if lateral variability could be delineated and used to group borings for development of specific FIRS in conformance with the US-APWR Standard Design. As indicated by the statistical variability (defined as the standard deviation about the mean), the velocity measurements across the site for each individual engineering layer are very consistent. Therefore, a single seismic velocity profile is warranted for the shallow subsurface.

The elevation ranges for velocity layer boundaries match closely to stratigraphic engineering layers described in Subsection 2.5.4.3. Throughout the Glen Rose Formation, shale layers exhibit markedly lower velocities than limestone layers, and the interfaces between these strata are typically sharp and accompanied by strong velocity changes (Figure 2.5.4-239). Typical shale S-wave average velocities are about 2,500 to 3,000 fps. In contrast, typical limestone S-wave average velocities are about 3,500 to 7,000 fps. Seismic velocities exhibit a uniform decrease in Twin Mountain Formation engineering layers F through I (weakly cemented sandstone, shale, and claystone), irrespective of lithology. These layers exhibit a relatively small range in S-wave average velocity, about 3,000 to 3,500 fps. Seismic velocity increases substantially within the

Plant grade is directly underlain by Glen Rose Formation limestone of engineering Layer A around the CPNPP Units 3 and 4 power block, and seismic category I and II structures, with the exception of the UHS, as shown in Figure 2.5.4-215. Foundation support for all seismic category I and II structures and power block is provided by a level, cleaned excavated surface in Glen Rose Formation limestone of engineering Layer C, as described in Subsection 2.5.4.3 and shown diagrammatically in Figure 2.5.4-217. Layer C is massive, competent limestone with an average thickness of 60 ft. Layer C and underlying Glen Rose Formation Layers D through F (primarily massive limestone with thin shale intervals), are indurated rock materials of Late Cretaceous age that are not susceptible to significant seismically induced strength degradation, particularly at the low level of seismic strain associated with the GMRS ground motions. As a result, static properties measured for Glen Rose Formation rock are reflective of anticipated seismic response (Subsection 2.5.2). Any required overexcavation below seismic category I and II foundation basemat elevations to reach the Layer C limestone are backfilled with fill concrete that is equal to, or stiffer than, the Glen Rose Formation rock layers (Table 2.5.2-227).

Dynamic shear modulus reduction (G/G_{max}) and damping properties for rock strata are developed based on field seismic velocity measurements summarized in Subsection 2.5.4.4.2 and laboratory-determined static properties described in Subsection 2.5.4.2. Best estimate values for both shear modulus and damping are provided for each layer in Table 2.5.2-227, and consider essentially linear response within the seismic strain ranges. As discussed in Subsection 2.5.4.3, the rock strata are horizontal to near-horizontal, and lateral variability in rock properties within each stratum is very low. Therefore, a single set of G/G_{max} and damping curves is justified and can be applied for the site seismic evaluation. Lower bound shear modulus for site rock strata ranges between 110.1 (shale) ksi and 879.1 (limestone) ksi. Upper bound shear modulus for rock strata ranges between 317.1 (shale) ksi and 2,531.7 (limestone) ksi. Low strain damping values range between 1.8 and 2.0 percent, and are based on in situ geophysical borehole seismic velocity measurements for the shallow velocity profile discussed in Subsection 2.5.4.4.2. Low strain damping values for the deep velocity profile below the maximum depth of borehole testing are based on linear extrapolation of velocity and lithologic matching from the shallow profile.

The GMRS and FIRS analysis profiles consider fill concrete between the base of the seismic category I and II structural foundation mats and the top of Glen Rose Formation engineering Layer C. Dynamic modulus values ranging between 748.0 ksi (lower bound) and 2,991.8 ksi (upper bound) for fill concrete are based on an assumed mean S-wave velocity of about 6,400 fps and an approximate wet unit weight of 150 pcf for typical concrete, meeting the specification discussed in Subsection 2.5.4.5.4.1.2.

Although no seismic category I or II structures ~~except the electrical duct banks~~ are supported by engineered fill, limited compacted backfill is placed against the lower structural walls between subgrade and plant yard elevations. Dynamic properties for compacted backfill listed in Table 2.5.2-227 are derived based on standard RCOL2_03.08.04-63

EPRI (1993) (Reference 2.5-387) shear modulus reduction and damping curves for granular fill. Subsection 2.5.4.5.4.1.1 discusses compacted backfill requirements, including the use of granular material. Fill specifications are generally consistent with the specifications and the fill placed at CPNPP Units 1 and 2, and are derived either from processing of on-site excavation materials, or commercial quarries in the site vicinity. Compacted backfill is divided into three basic layers: a surface layer from plant grade to a depth of 3 ft; a shallow layer from 3 ft to 20 ft depth below plant grade; and, a deeper layer between the depths of 20 ft and 40 ft below plant grade. Different EPRI curves are used for the fill less than 20 ft deep and greater than 20 ft deep. Shear modulus and damping values are based on assumed mean S-wave velocities of 650 fps for surface fill, 800 fps for shallow fill, and 1000 fps for deeper fill, Poisson's ratio of 0.35, and wet unit weight of 125 pcf. Based on a minimum shear modulus variation factor (C_V) of 1.0, the Upper and Lower bound ranges for shear moduli for compacted fill are between 5.7 ksi and 22.8 ksi for surface fill, between 8.7 ksi and 34.6 ksi for fill between 3 ft and 20 ft deep, and between 13.5 ksi and 54.0 ksi for fill greater than 20 ft deep. The broad range between Lower and Upper Bound values accommodates significant variation in fill properties that are larger than typically achieved by controlled fill materials and placement specified in Subsection 2.5.4.5.4.1.1. This approach conservatively captures reasonable ranges for fill properties. Low-strain damping ratios are assigned as 1.5 percent for fill less than and equal to 20 ft deep, and 1.1 percent for fill deeper than 20 ft. EPRI-based (Reference 2.5-387) shear modulus reduction and damping curves for the compacted fill are shown on Figure 2.5.2-232.

~~Verification of the seismic S-wave velocity of the compacted fill material placed underneath the seismic category I duct banks is required to confirm that the actual S-wave velocity values of the backfill materials are within the above described variability.~~ RCOL2_03.08.04-63

2.5.4.8 Liquefaction Potential

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.8 with the following.

In accordance with the requirements of 10 CFR Parts 50 and 100, an analysis of soil liquefaction potential was performed for soils adjacent to and under the seismic category I and II structures according to guidelines provided in RG 1.198. US-APWR Key Site Parameters (DCD Table 2.0-1) allows no liquefaction potential for seismic category I structures.

Soil materials that are considered to be susceptible to liquefaction include loose saturated sands and non-plastic silts. Liquefaction is typically restricted to Holocene and late-Pleistocene age alluvial soils and hydraulically-placed sand fill in areas of moderate to high seismicity. The site is an area of very low seismicity. The results of the ground motion and site response analysis indicate that the peak ground acceleration (PGA) ranges between 0.045g and 0.07g.

All seismic category I and power block structures associated with Units 3 and 4 are founded on stable Glen Rose Formation limestone Layer C, as discussed in Subsection 2.5.4.3. The Glen Rose Formation rock is late Cretaceous in age, indurated, and not susceptible to liquefaction. As discussed in Subsection 2.5.4.1, no paleoseismic evidence of past liquefaction was observed at the site, or is documented within the 25 mi radius region surrounding the site.

The foundation base mats of all seismic category I and II structures are founded on a limestone layer (engineering Layer C)~~, with the exception of seismic category I duct banks that are embedded in compacted fill adjcent to the nuclear island~~. RCOL2_03.08.04-63

The fill materials placed within the excavated areas around Units 3 and 4 and in the north-facing fill slopes are not considered prone to liquefaction for the following reasons:

- All fill material consists of engineered compacted fill with a minimum relative compaction of 95 percent (ASTM D1557). The corrected/ normalized standard penetration test N-Values are expected to be higher than 30 blows per foot, which is outside the range considered susceptible to soil liquefaction.

- The engineered compacted fill materials are not in a saturated state. The permanent groundwater table is well below the engineered compacted fill malerials.

- To minimize any potential for buildup of hydrostatic pressures within the engineered compacted fill, adequate drainage is provided for all below-grade structures and retaining walls, and at the base of all fill slopes.

Thus, the engineered compacted fill does not meet the conditions stated in RG 1.206 or RG 1.198 that would cause suspicion of a potential for liquefaction, and no liquefaction analysis is necessary. Even in the unlikely event that the engineered compacted fill became completely saturated, the soil density is too high and the site PGA range is too low to suspect a potential for liquefaction.Liquefaction is therefore not a hazard to CPNPP Units 3 and 4 seismic category I or major plant structures, and the site characteristics meet the US-APWR Standard Design criteria.

Soil liquefaction is also not anticipated within the engineered compacted fill surrounding Units 3 and 4 structures because 1) the permanent groundwater is below the lowest elevation of fill and 2) fill is placed with a high degree of material control and compaction, and 3) the CPNPP site is an area of low seismicity with low GMRS design motions, as described in Subsection 2.5.2.

2.5.4.9 Earthquake Site Characteristics

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.9 with the following.

Table 2.5.2-236
1E-5 and GMRS Amplitudes for GMRS Elevation, Horizontal and Vertical

RCOL2_03.08.04-72

~~Horizontal and vertical a~~Amplitudes for GMRS elevation			
Frequency (Hz)	Horizontal 1E-5 UHRS (g)	Hoizontal GMRS (g)	Vertical GMRS (g)
100	~~7.43E-02~~8.26E-02	~~3.34E-02~~3.72-E-02	3.72E-02
90	~~7.48E-02~~8.33E-02	~~3.37E-02~~3.75E-02	3.75E-02
80	~~7.54E-02~~8.42E-02	~~3.39E-02~~3.79E-02	3.79E-02
75	~~7.57E-02~~8.46E-02	~~3.41E-02~~3.81E-02	3.81E-02
70	~~7.61E-02~~8.51E-02	~~3.42E-02~~3.83E-02	3.83E-02
60	~~7.68E-02~~8.62E-02	~~3.46E-02~~3.88E-02	3.88E-02
50	~~7.78E-02~~8.76E-02	~~3.50E-02~~3.94E-02	3.94E-02
40	~~7.89E-02~~8.92E-02	~~3.55E-02~~4.01E-02	4.01E-02
30	~~8.04E-02~~9.14E-02	~~3.62E-02~~4.11E-02	4.11E-02
25	~~8.14E-02~~9.28E-02	~~3.66E-02~~4.18E-02	4.18E-02
20	~~8.53E-02~~9.74E-02	~~3.84E-02~~4.38E-02	4.38E-02
15	~~9.05E-02~~1.04-E01	~~4.07E-02~~4.66E-02	4.66E-02
12.5	~~9.40E-02~~1.08E-01	~~4.23E-02~~4.85E-02	4.85E-02
10	~~9.85E-02~~1.13E-01	~~4.43E-02~~5.09E-02	5.09E-02
9	~~9.99E-02~~1.14E-01	~~4.49E-02~~5.14E-02	5.14E-02
8	~~1.01E-01~~1.16E-01	~~4.57E-02~~5.20E-02	5.20E-02
7.5	~~1.02E-01~~1.16E-01	~~4.61E-02~~5.23E-02	5.23E-02
7	~~1.03E-01~~1.17E-01	~~4.65E-02~~5.27E-02	5.27E-02
6	~~1.05E-01~~1.19E-01	~~4.74E-02~~5.35E-02	5.34E-02
5	~~1.08E-01~~1.21E-01	~~4.86E-02~~5.45E-02	5.44E-02
4	~~1.31E-01~~1.42E-01	~~5.91E-02~~6.39E-02	6.38E-02
3	~~1.51E-01~~1.58E-01	~~6.78E-02~~7.13E-02	6.11E-02
2.5	~~1.55E-01~~1.62E-01	~~6.98E-02~~7.29E-02	5.21E-02
2	~~1.55E-01~~1.54E-01	~~6.99E-02~~6.94E-02	4.93E-02
1.8	~~1.54E-01~~1.50E-01	~~6.92E-02~~6.75E-02	4.78E-02
1.5	~~1.43E-01~~1.36E-01	~~6.42E-02~~6.14E-02	4.32E-02
1.25	~~1.28E-01~~1.20E-01	~~5.76E-02~~5.41E-02	3.79E-02
1	~~1.09E-01~~1.00E-01	~~4.91E-02~~4.50E-02	3.13E-02
0.9	~~1.08E-01~~9.65E-02	~~4.86E-02~~4.34E-02	3.01E-02
0.8	~~1.07E-01~~9.27E-02	~~4.80E-02~~4.17E-02	2.88E-02
0.7	~~1.05E-01~~8.85E-02	~~4.74E-02~~3.98E-02	2.74E-02
0.6	~~1.04E-01~~8.40E-02	~~4.67E-02~~3.78E-02	2.59E-02
0.5	~~1.02E-01~~7.89E-02	~~4.59E-02~~3.55E-02	2.42E-02
0.4	~~8.16E-02~~6.13E-02	~~3.67E-02~~2.76E-02	1.87E-02
0.3	~~6.12E-02~~4.19E-02	~~2.75E-02~~1.89E-02	1.27E-02
0.2	~~4.08E-02~~2.03E-02	~~1.84E-02~~9.12E-03	6.09E-03
0.15	~~3.06E-02~~1.14E-02	~~1.38E-02~~5.11E-03	3.42E-03
0.125	~~2.55E-02~~7.84E-03	~~1.15E-02~~3.53E-03	2.63E-03
0.1	~~1.63E-02~~4.95E-03	~~7.34E-03~~2.23E-03	1.49E-03

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RCOL2_02.05.02-16 S02

Table 2.5.2-237 (Sheet 1 of 3)
1E-5 and FIRS Amplitudes for FIRS Elevations, Horizontal and Vertical

RCOL2_02.05.02-16 S01

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	FIRS2 (g)		~~FIRS3~~	FIRS~~4~~3 (g)			~~FIRS4 CoV50~~	
Frequency (Hz)	1E-5 UHRS	Horizontal FIRS2	~~1E-5 UHRS~~ Vertical FIRS2	~~FIRS3~~ 1E-5 UHRS	~~1E-5 UHRS~~ Horizontal FIRS 3	Vertical FIRS~~4~~3	~~1E-5 UHRS~~	~~FIRS4 CoV50~~
100	~~3.31~~8.49E-02	~~1.49~~3.82E-02	~~3.73E-02~~ 3.82E-02	~~1.68E-02~~ 1.01E-01	~~5.45E-02~~ 4.55E-02	~~2.45E-02~~ 4.55E-02	~~5.45E-02~~	~~2.45E-02~~
90	~~3.33~~8.58E-02	~~1.50~~3.86E-02	~~3.79E-02~~ 3.86E-02	~~1.71E-02~~ 1.03E-01	~~5.53E-02~~ 4.64E-02	~~2.49E-02~~ 4.64E-02	~~5.53E-02~~	~~2.49E-02~~
80	~~3.35~~8.69E-02	~~1.51~~3.91E-02	~~3.86E-02~~ 3.91E-02	~~1.74E-02~~ 1.05E-01	~~5.63E-02~~ 4.75E-02	~~2.53E-02~~ 4.75E-02	~~5.62E-02~~	~~2.53E-02~~
75	~~3.36~~8.75E-02	~~1.51~~3.94E-02	~~3.90E-02~~ 3.94E-02	~~1.76E-02~~ 1.07E-01	~~5.68E-02~~ 4.80E-02	~~2.56E-02~~ 4.80E-02	~~5.67E-02~~	~~2.55E-02~~
70	~~3.38~~8.81E-02	~~1.52~~3.96E-02	~~3.95E-02~~ 3.96E-02	~~1.78E-02~~ 1.08E-01	~~5.74E-02~~ 4.87E-02	~~2.58E-02~~ 4.87E-02	~~5.72E-02~~	~~2.57E-02~~
60	~~3.40~~8.95E-02	~~1.53~~4.03E-02	~~4.04E-02~~ 4.03E-02	~~1.82E-02~~ 1.11E-01	~~5.87E-02~~ 5.02E-02	~~2.64E-02~~ 5.02E-02	~~5.84E-02~~	~~2.63E-02~~
50	~~3.44~~9.12E-02	~~1.55~~4.10E-02	~~4.16E-02~~ 4.10E-02	~~1.87E-02~~ 1.15E-01	~~6.02E-02~~ 5.20E-02	~~2.71E-02~~ 5.20E-02	~~5.98E-02~~	~~2.69E-02~~
40	~~3.48~~9.33E-02	~~1.57~~4.20E-02	~~4.31E-02~~ 4.20E-02	~~1.94E-02~~ 1.21E-01	~~6.22E-02~~ 5.42E-02	~~2.80E-02~~ 5.42E-02	~~6.17E-02~~	~~2.77E-02~~
30	~~3.53~~9.62E-02	~~1.59~~4.33E-02	~~4.51E-02~~ 4.33E-02	~~2.03E-02~~ 1.27E-01	~~6.49E-02~~ 5.73E-02	~~2.92E-02~~ 5.73E-02	~~6.41E-02~~	~~2.88E-02~~
25	~~3.57~~9.80E-02	~~1.61~~4.41E-02	~~4.64E-02~~ 4.41E-02	~~2.09E-02~~ 1.32E-01	~~6.66E-02~~ 5.94E-02	~~3.00E-02~~ 5.94E-02	~~6.57E-02~~	~~2.96E-02~~
20	~~3.74~~1.03E-~~02~~01	~~1.68~~4.63E-02	~~5.32E-02~~ 4.63E-02	~~2.40E-02~~ 1.55E-01	~~7.33E-02~~ 6.96E-02	~~3.30E-02~~ 6.96E-02	~~7.35E-02~~	~~3.31E-02~~
15	~~3.98~~1.10E-~~02~~01	~~1.79~~4.94E-02	~~6.11E-02~~ 4.94E-02	~~2.75E-02~~ 1.93E-01	~~8.29E-02~~ 8.67E-02	~~3.73E-02~~ 8.67E-02	~~8.49E-02~~	~~3.82E-02~~
12.5	~~4.13~~1.14E-~~02~~01	~~1.86~~5.14E-02	~~6.54E-02~~ 5.14E-02	~~2.94E-02~~ 2.02E-01	~~8.96E-02~~ 9.10E-02	~~4.03E-02~~ 9.10E-02	~~9.30E-02~~	~~4.18E-02~~
10	~~4.33~~1.20E-~~02~~01	~~1.95~~5.40E-02	~~6.98E-02~~ 5.40E-02	~~3.14E-02~~ 2.08E-01	~~9.86E-02~~ 9.36E-02	~~4.44E-02~~ 9.36E-02	~~1.04E-01~~	~~4.68E-02~~
9	~~4.38~~1.21E-~~02~~01	~~1.97~~5.43E-02	~~6.77E-02~~ 5.43E-02	~~3.04E-02~~ 1.99E-01	~~1.11E-01~~ 8.97E-02	~~5.00E-02~~ 8.97E-02	~~1.12E-01~~	~~5.04E-02~~
8	~~4.44~~1.22E-~~02~~01	~~2.00~~5.47E-02	~~6.52E-02~~ 5.47E-02	~~2.93E-02~~ 1.89E-01	~~1.24E-01~~ 8.52E-02	~~5.59E-02~~ 8.52E-02	~~1.20E-01~~	~~5.42E-02~~
7.5	~~4.47~~1.22E-~~02~~01	~~2.01~~5.49E-02	~~6.38E-02~~ 5.49E-02	~~2.87E-02~~ 1.84E-01	~~1.31E-01~~ 8.27E-02	~~5.90E-02~~ 8.27E-02	~~1.25E-01~~	~~5.61E-02~~
7	~~4.51~~1.23E-~~02~~01	~~2.03~~5.51E-02	~~6.23E-02~~ 5.51E-02	~~2.80E-02~~ 1.78E-01	~~1.38E-01~~ 8.01E-02	~~6.20E-02~~ 8.00E-02	~~1.29E-01~~	~~5.80E-02~~
6	~~4.59~~1.24E-~~02~~01	~~2.06~~5.56E-02	~~5.88E-02~~ 5.56E-02	~~2.65E-02~~ 1.65E-01	~~1.52E-01~~ 7.41E-02	~~6.83E-02~~ 7.41E-02	~~1.37E-01~~	~~6.18E-02~~
5	~~4.68~~1.25E-~~02~~01	~~2.11~~5.63E-02	~~5.45E-02~~ 5.62E-02	~~2.45E-02~~ 1.49E-01	~~1.65E-01~~ 6.71E-02	~~7.43E-02~~ 6.70E-02	~~1.45E-01~~	~~6.53E-02~~
4	~~5.51~~1.43E-~~02~~01	~~2.48~~6.44E-02	~~6.33E-02~~ 6.44E-02	~~2.85E-02~~ 1.66E-01	~~1.48E-01~~ 7.45E-02	~~6.64E-02~~ 7.44E-02	~~1.40E-01~~	~~6.32E-02~~
3	~~6.17~~1.57E-~~02~~01	~~2.77~~7.06E-02	~~7.01E-02~~ 6.05E-02	~~3.16E-02~~ 1.77E-01	~~1.24E-01~~ 7.95E-02	~~5.57E-02~~ 6.81E-02	~~1.30E-01~~	~~5.87E-02~~

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Table 2.5.2-237 (Sheet 2 of 3)
1E-5 and FIRS Amplitudes for FIRS Elevations, Horizontal and Vertical

2.5	~~6.28~~1.59E-~~02~~01	~~2.83~~7.16E-02	~~7.11E-02~~ 5.12E-02	~~3.20E-02~~ 1.77E-01	~~1.08E-01~~ 7.97E-02	~~4.84E-02~~ 5.69E-02	~~1.21E-01~~	~~5.45E-02~~
2	~~6.49~~1.55E-~~02~~01	~~2.92~~6.97E-02	~~7.34E-02~~ 4.95E-02	~~3.30E-02~~ 1.73E-01	~~1.08E-01~~ 7.79E-02	~~4.84E-02~~ 5.53E-02	~~1.18E-01~~	~~5.31E-02~~
1.8	~~6.50~~1.52E-~~02~~01	~~2.93~~6.84E-02	~~7.34E-02~~ 4.85E-02	~~3.30E-02~~ 1.70E-01	~~1.05E-01~~ 7.65E-02	~~4.71E-02~~ 5.42E-02	~~1.14E-01~~	~~5.12E-02~~
1.5	~~6.13~~1.40E-~~02~~01	~~2.76~~6.30E-02	~~6.91E-02~~ 4.44E-02	~~3.11E-02~~ 1.57E-01	~~9.65E-02~~ 7.06E-02	~~4.34E-02~~ 4.97E-02	~~1.04E-01~~	~~4.66E-02~~
1.25	~~5.57~~1.25E-~~02~~01	~~2.51~~5.62E-02	~~6.27E-02~~ 3.93E-02	~~2.82E-02~~ 1.40E-01	~~8.61E-02~~ 6.30E-02	~~3.87E-02~~ 4.41E-02	~~9.13E-02~~	~~4.11E-02~~
1	~~4.80~~1.05E-~~02~~01	~~2.16~~4.73E-02	~~5.40E-02~~ 3.29E-02	~~2.43E-02~~ 1.18E-01	~~7.29E-02~~ 5.31E-02	~~3.28E-02~~ 3.70E-02	~~7.65E-02~~	~~3.44E-02~~
0.9	~~4.75~~1.01E-~~02~~01	~~2.14~~4.56E-02	~~5.33E-02~~ 3.16E-02	~~2.40E-02~~ 1.15E-01	~~7.14E-02~~ 5.16E-02	~~3.21E-02~~ 3.58E-02	~~7.46E-02~~	~~3.36E-02~~
0.8	~~4.69~~9.73E-02	~~2.11~~4.38E-02	~~5.25E-02~~ 3.03E-02	~~2.36E-02~~ 1.11E-01	~~6.97E-02~~ 4.99E-02	~~3.14E-02~~ 3.45E-02	~~7.25E-02~~	~~3.26E-02~~
0.7	~~4.63~~9.30E-02	~~2.08~~4.19E-02	~~5.16E-02~~ 2.88E-02	~~2.32E-02~~ 1.07E-01	~~6.79E-02~~ 4.81E-02	~~3.06E-02~~ 3.31E-02	~~7.02E-02~~	~~3.16E-02~~
0.6	~~4.55~~8.83E-02	~~2.05~~3.97E-02	~~5.06E-02~~ 2.72E-02	~~2.28E-02~~ 1.02E-01	~~6.58E-02~~ 4.60E-02	~~2.96E-02~~ 3.16E-02	~~6.77E-02~~	~~3.05E-02~~
0.5	~~4.47~~8.30E-02	~~2.01~~3.74E-02	~~4.95E-02~~ 2.55E-02	~~2.23E-02~~ 9.72E-02	~~6.35E-02~~ 4.37e-02	~~2.86E-02~~ 2.98E-02	~~6.48E-02~~	~~2.92E-02~~
0.4	~~3.58~~6.35E-02	~~1.61~~2.86E-02	~~3.96E-02~~ 1.94E-02	~~1.78E-02~~ 7.34E-02	~~5.08E-02~~ 7.34E-02	~~2.29E-02~~ 2.24E-02	~~5.18E-02~~	~~2.33E-02~~
0.3	~~2.68~~4.23E-02	~~1.21~~1.90E-02	~~2.97E-02~~ 1.28E-02	~~1.34E-02~~ 4.96E-02	~~3.81E-02~~4.96E-02	~~1.71E-02~~ 1.50E-02	~~3.89E-02~~	~~1.75E-02~~
0.2	~~1.79~~2.03E-02	~~8.05~~9.13E-03	~~1.98E-02~~ 6.10E-03	~~8.91E-03~~ 2.45E-02	~~2.54E-02~~ 2.45E-02	~~1.14E-02~~ 7.37E-03	~~2.59E-02~~	~~1.17E-02~~
0.15	~~1.34~~1.15E-02	~~6.03~~5.17E-03	~~1.49E-02~~ 3.46E-03	~~6.68E-03~~ 1.39E-02	~~1.91E-02~~ 1.39E-02	~~8.57E-03~~ 4.19E-03	~~1.94E-02~~	~~8.75E-03~~
0.125	~~1.12~~7.97E-~~02~~03	~~5.03~~3.58E-03	~~1.24E-02~~ 2.40E-03	~~5.57E-03~~ 9.62E-03	~~1.59E-02~~ 9.62E-03	~~7.14E-03~~ 2.89E-03	~~1.62E-02~~	~~7.29E-03~~
0.1	~~7.15~~5.05E-03	~~3.22~~2.27E-03	~~7.92E-03~~ 1.52E-03	~~3.56E-03~~ 6.09E-03	~~1.02E-02~~ 6.09E-03	~~4.57E-03~~ 1.83E-03	~~1.04E-02~~	~~4.67E-03~~

Table 2.5.2-237 (Sheet 3 of 3)
1E-5 and FIRS Amplitudes for FIRS Elevations, Horizontal and Vertical

RCOL2_03.08.04-72

	FIRS4 (g)			FIRS4-CoV50 (g)				
Frequency (Hz)	1E-5 UHRS	Horizontal FIRS4	Vertical FIRS4	1E-5 UHRS	Horizontal FIRS4-CoV50	Vertical FIRS4-CoV50		
100	1.51E-01	6.80E-02	6.80E-02	1.48E-01	6.66E-02	6.66E-02		
90	1.54E-01	6.93E-02	6.93E-02	1.51E-01	6.79E-02	6.79E-02		
80	1.57E-01	7.07E-02	7.07E-02	1.54E-01	6.93E-02	6.93E-02		
75	1.59E-01	7.16E-02	7.16E-02	1.56E-01	7.01E-02	7.01E-02		
70	1.61E-01	7.25E-02	7.25E-02	1.58E-01	7.10E-02	7.10E-02		
60	1.66E-01	7.45E-02	7.45E-02	1.62E-01	7.30E-02	7.30E-02		
50	1.71E-01	7.70E-02	7.70E-02	1.67E-01	7.54E-02	7.54E-02		
40	1.78E-01	8.02E-02	8.02E-02	1.74E-01	7.84E-02	7.84E-02		
30	1.88E-01	8.45E-02	8.45E-02	1.83E-01	8.25E-02	8.25E-02		
25	1.94E-01	8.73E-02	8.73E-02	1.90E-01	8.53E-02	8.53E-02		
20	2.14E-01	9.61E-02	9.61E-02	2.13E-01	9.60E-02	9.60E-02		
15	2.42E-01	1.09E-01	1.09E-01	2.49E-01	1.12E-01	1.12E-01		
12.5	2.62E-01	1.18E-01	1.18E-01	2.74E-01	1.23E-01	1.23E-01		
10	2.88E-01	1.30E-01	1.30E-01	3.08E-01	1.39E-01	1.39E-01		
9	3.25E-01	1.46E-01	1.46E-01	3.30E-01	1.48E-01	1.48E-01		
8	3.64E-01	1.64E-01	1.64E-01	3.52E-01	1.58E-01	1.58E-01		
7.5	3.83E-01	1.73E-01	1.73E-01	3.62E-01	1.63E-01	1.63E-01		
7	4.03E-01	1.81E-01	1.81E-01	3.73E-01	1.68E-01	1.68E-01		
6	4.43E-01	1.99E-01	1.99E-01	3.94E-01	1.77E-01	1.77E-01		
5	4.80E-01	2.16E-01	2.16E-01	4.12E-01	1.85E-01	1.85E-01		
4	4.10E-01	1.84E-01	1.84E-01	3.83E-01	1.72E-01	1.72E-01		
3	3.23E-01	1.45E-01	1.25E-01	3.40E-01	1.53E-01	1.31E-01		
2.5	2.71E-01	1.22E-01	8.72E-02	3.08E-01	1.39E-01	9.91E-02		
2	2.58E-01	1.16E-01	8.25E-02	2.86E-01	1.29E-01	9.14E-02		
1.8	2.47E-01	1.11E-01	7.88E-02	2.71E-01	1.22E-01	8.64E-02		
1.5	2.23E-01	1.01E-01	7.08E-02	2.41E-01	1.08E-01	7.64E-02		
1.25	1.96E-01	8.81E-02	6.17E-02	2.08E-01	9.37E-02	6.57E-02		
1	1.62E-01	7.29E-02	5.07E-02	1.70E-01	7.64E-02	5.32E-02		
0.9	1.57E-01	7.07E-02	4.90E-02	1.64E-01	7.36E-02	5.11E-02		
0.8	1.52E-01	6.82E-02	4.72E-02	1.57E-01	7.06E-02	4.88E-02		
0.7	1.46E-01	6.56E-02	4.52E-02	1.50E-01	6.73E-02	4.64E-02		
0.6	1.39E-01	6.27E-02	4.30E-02	1.42E-01	6.37E-02	4.37E-02		
0.5	1.32E-01	5.94E-02	4.05E-02	1.33E-01	5.97E-02	4.07E-02		
0.4	1.03E-01	4.62E-02	3.13E-02	1.03E-01	4.61E-02	3.13E-02		
0.3	6.99E-02	3.15E-02	2.11E-02	6.95E-02	3.13E-02	2.10E-02		
0.2	3.34E-02	1.50E-02	1.00E-02	3.31E-02	1.49E-02	9.94E-03		
0.15	1.88E-02	8.47E-03	5.66E-03	1.86E-02	8.39E-03	5.61E-03		
0.125	1.30E-02	5.85E-03	3.91E-03	1.29E-02	5.80E-03	3.88E-03		
0.1	8.24E-03	3.71E-03	2.48E-03	8.17E-03	3.67E-03	2.46E-03		

RCOL2_02.05.02-16 S01
RCOL2_03.08.04-72

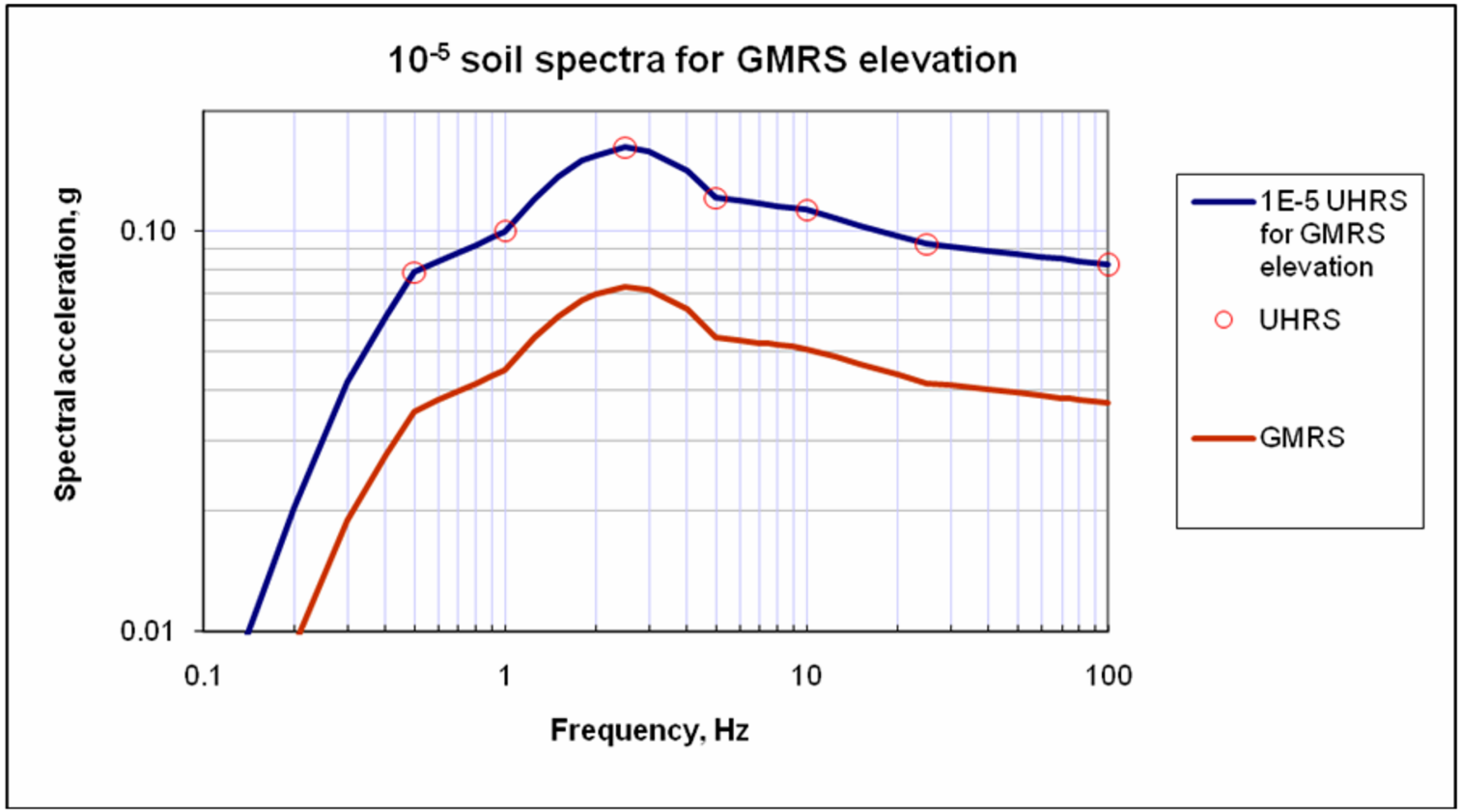

Figure 2.5.2-247 Comanche Peak 1E-5 UHRS (for GMRS Conditions) and GMRS, Horizontal ~~and Vertical~~

RCOL2_02.05.02-16 S01
RCOL2_03.08.04-72

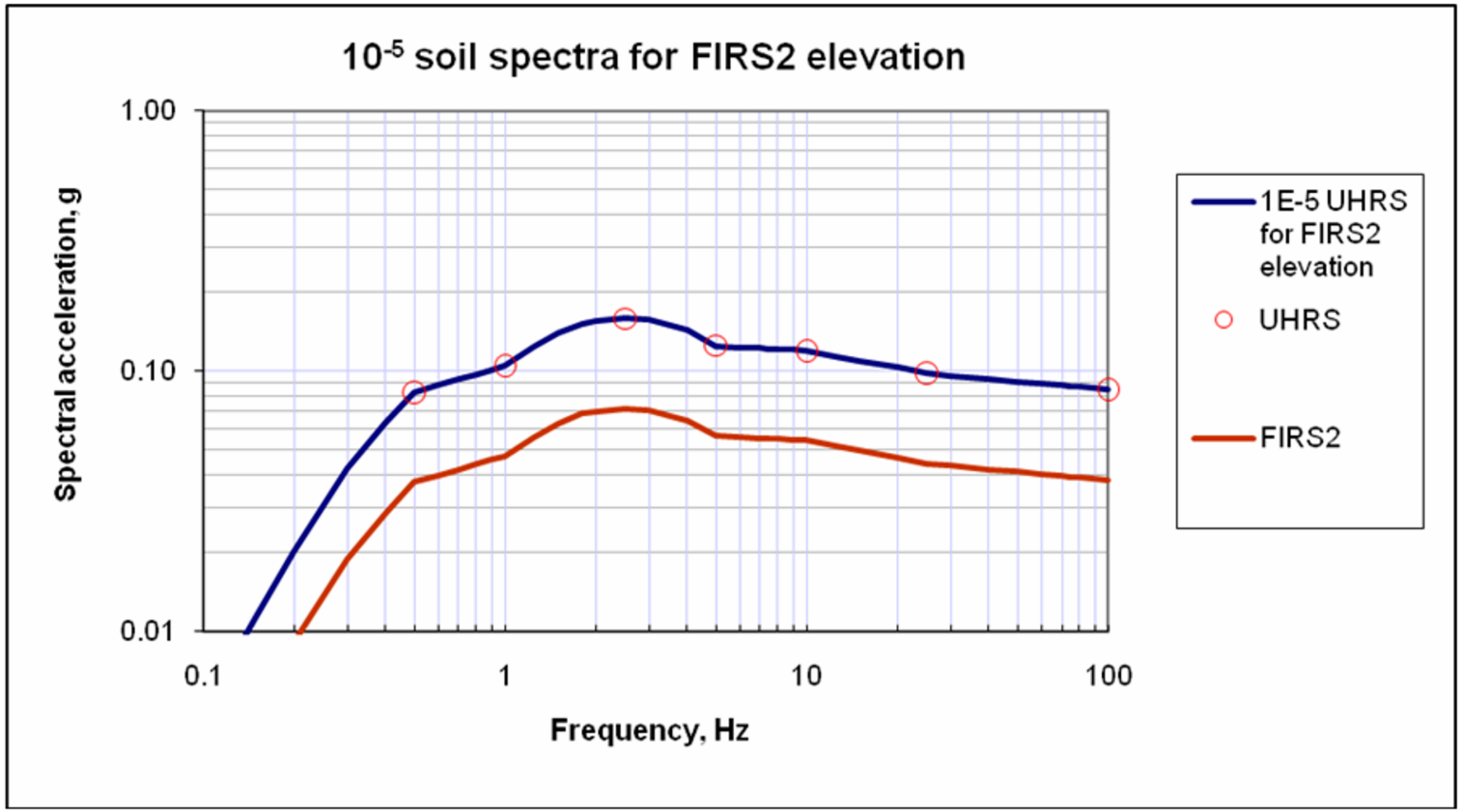

Figure 2.5.2-248 Comanche Peak 1E-5 UHRS (for FIRS2 Conditions) and FIRS2, Horizontal ~~and Vertical~~

RCOL2_02.05.02-16 S01
RCOL2_03.08.04-72

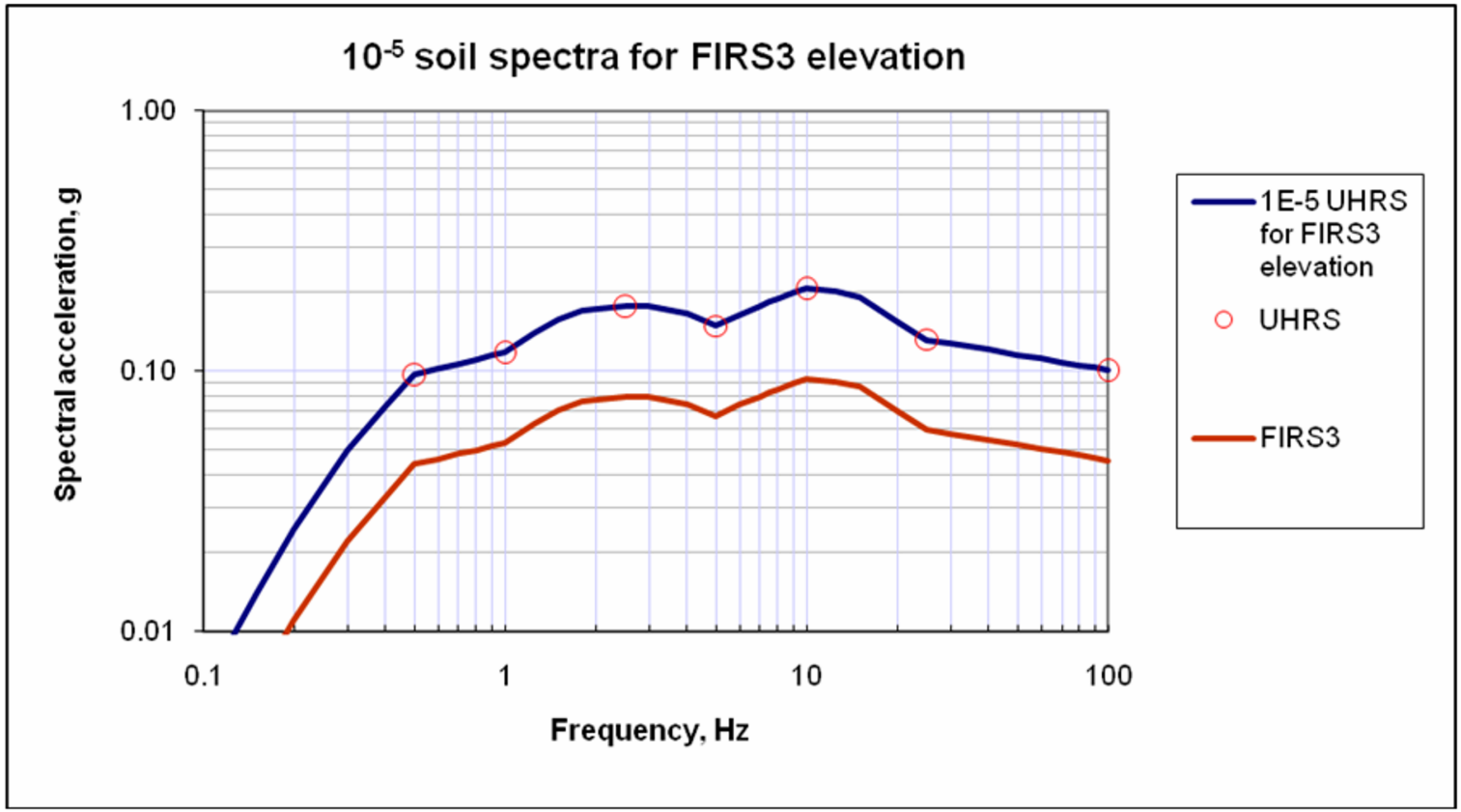

Figure 2.5.2-249 Comanche Peak 1E-5 UHRS (for FIRS3 Conditions) and FIRS3, Horizontal ~~and Vertical~~

RCOL2_02.05.02-16 S01
RCOL2_03.08.04-72

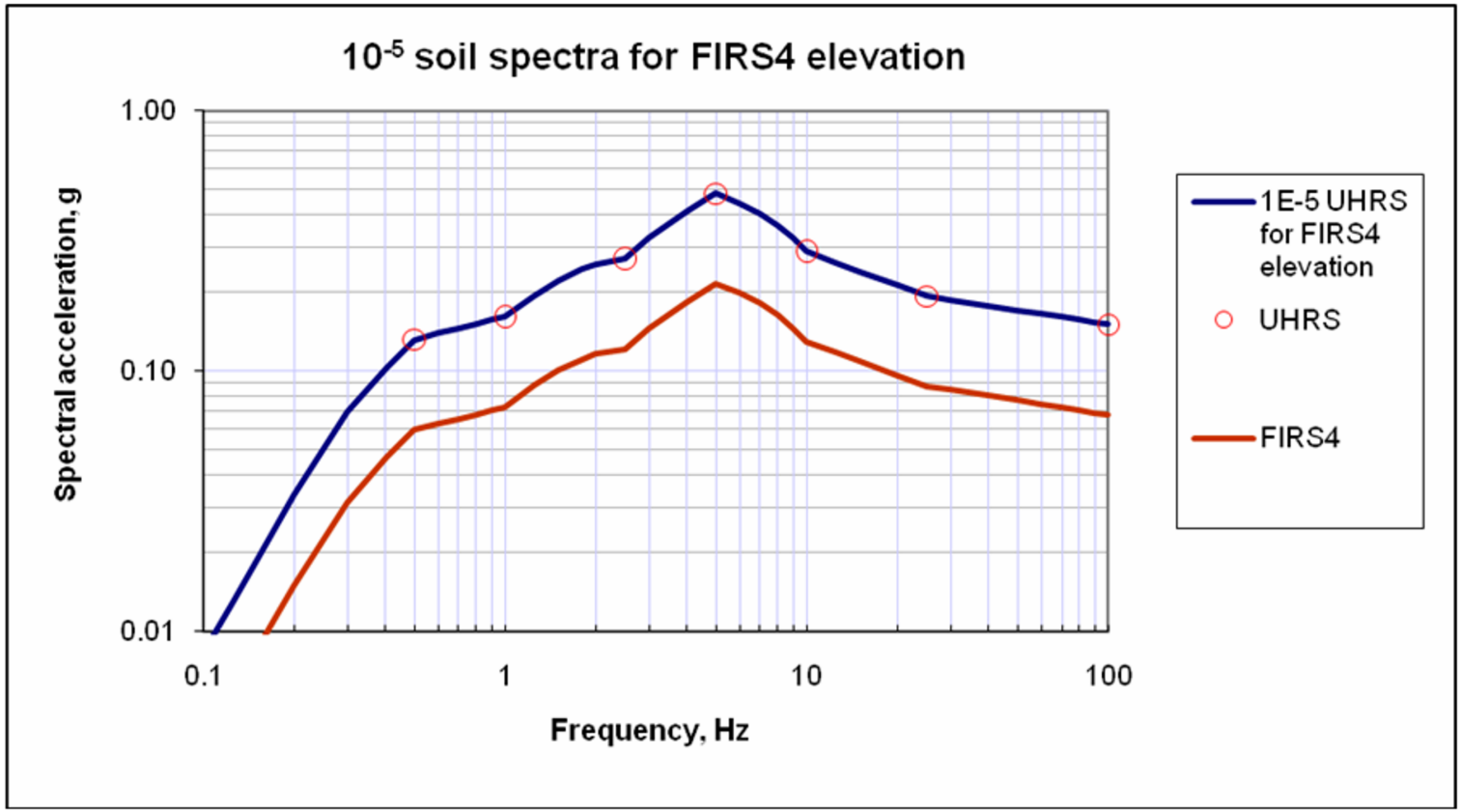

Figure 2.5.2-250 Comanche Peak 1E-5 UHRS (for FIRS4 Conditions) and FIRS4, Horizontal ~~and Vertical~~

RCOL2_02.05.02-16 S01
RCOL2_03.08.04-72

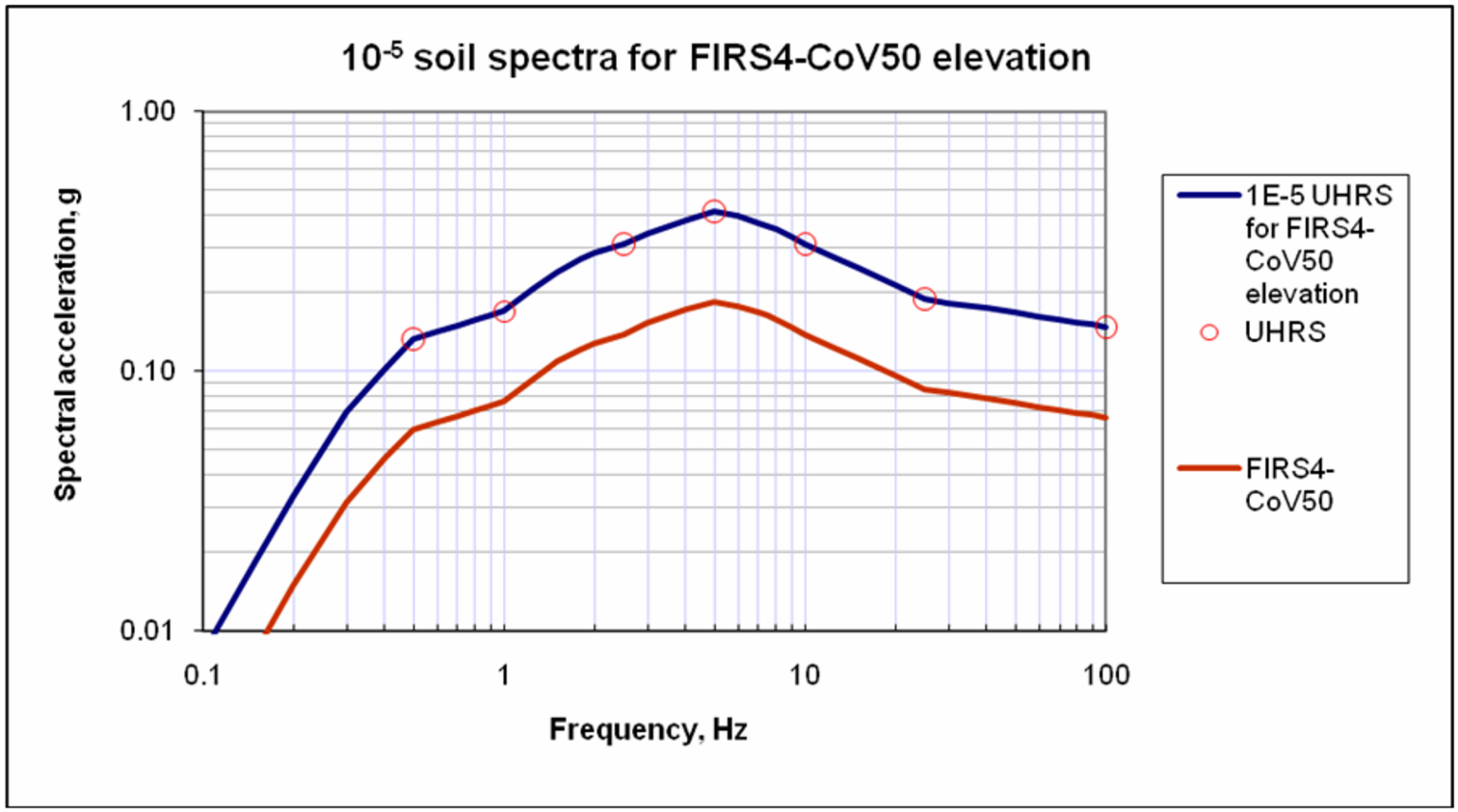

Figure 2.5.2-251 Comanche Peak 1E-5 UHRS (for FIRS4-CoV50 Conditions) and FIRS4-CoV50, Horizontal~~ and Vertical~~

RCOL2_03.08.04-72

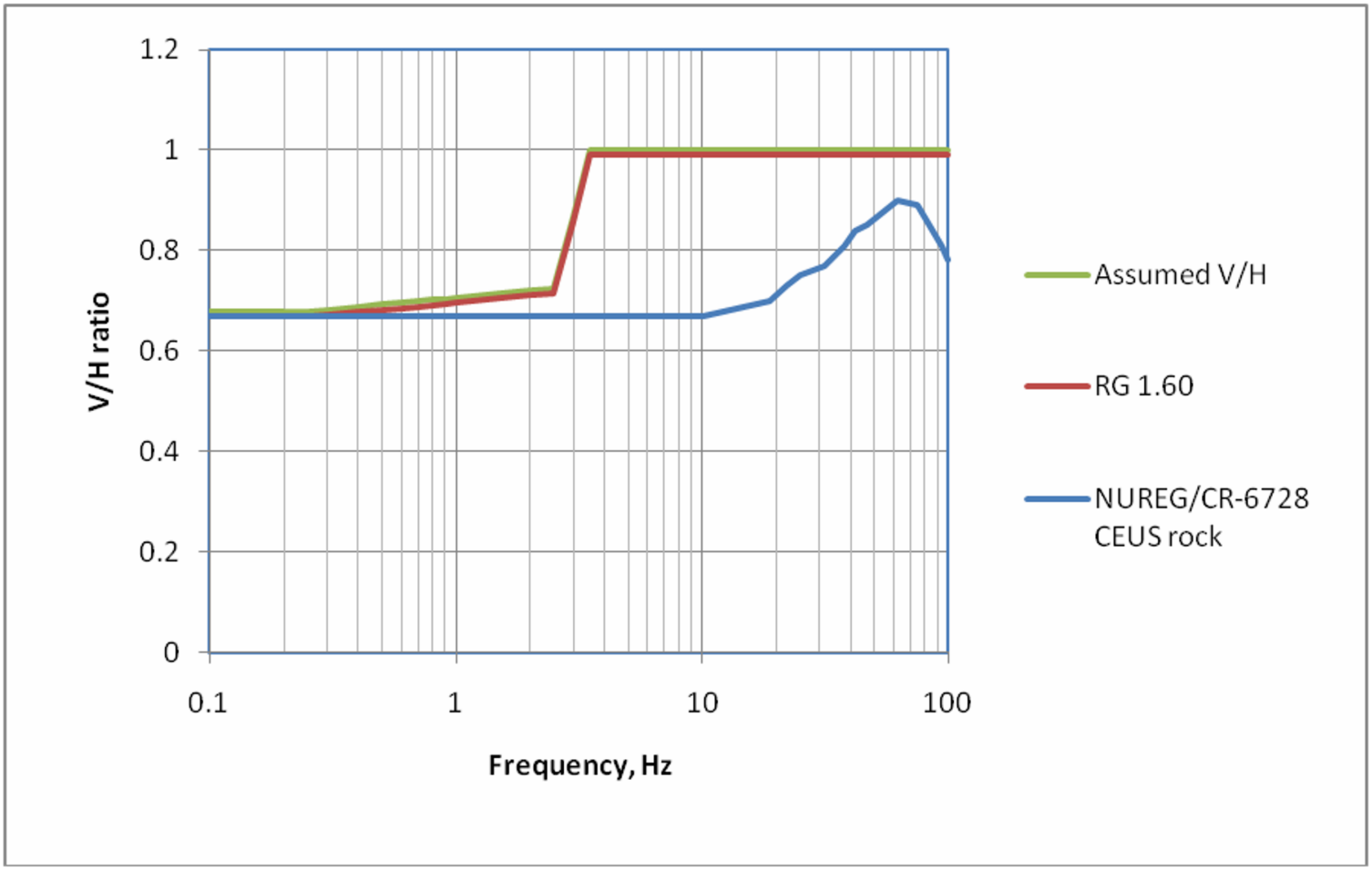

Figure 2.5.2-252 V/H Ratios from Two References, and Assumed V/H Ratio

RCOL2_03.08.04-72

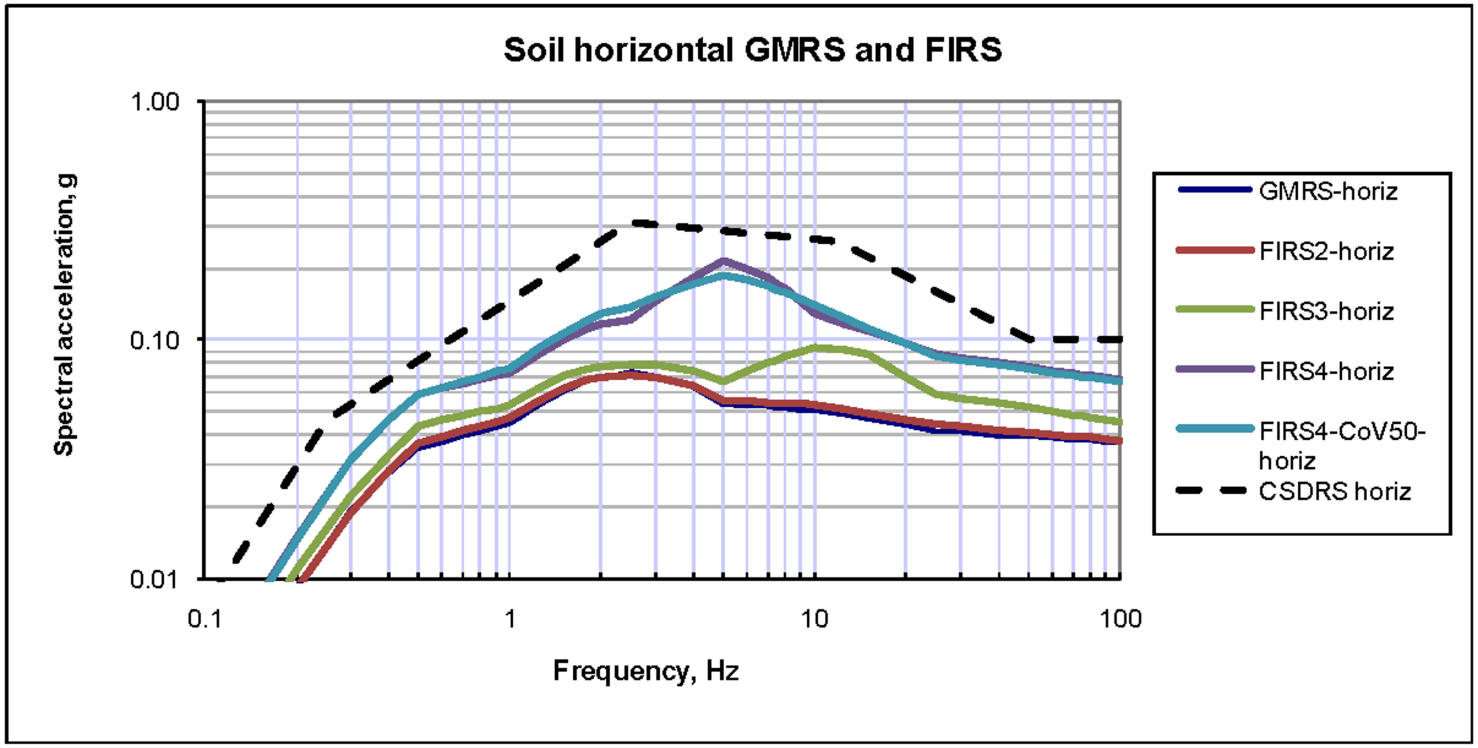

Figure 2.5.2-257 Horizontal GMRS and FIRS spectra plotted with minimum CSDRS spectra, 5% damping

RCOL2_03.08.04-72

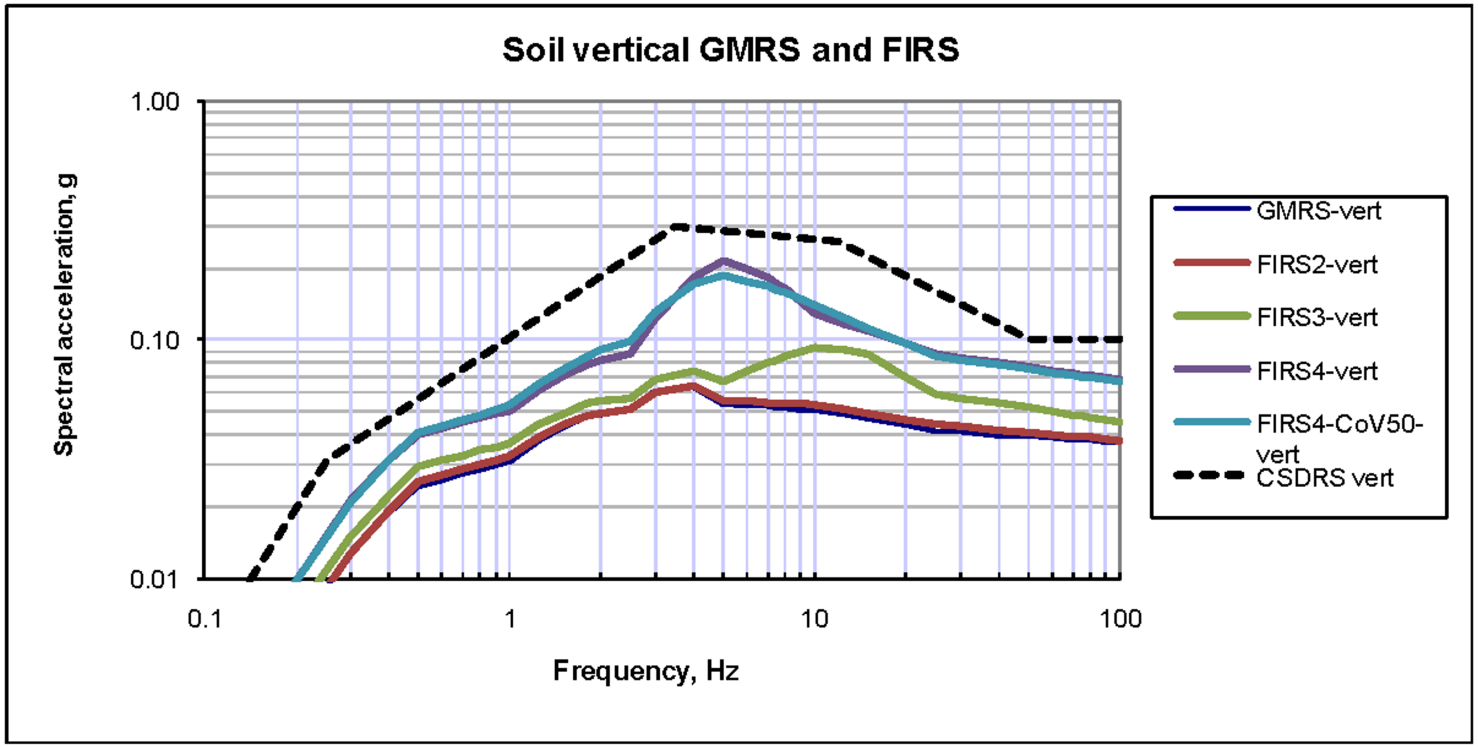

Figure 2.5.2-258 Vertical GMRS and FIRS spectra plotted with minimum CSDRS spectra, 5% damping

loading. As shown in FSAR Figures 3.8-206 through 3.8-211, the UHSRS complex is comprised of relatively low-rise, nearly rectangular structures that do not include any unusual or irregular geometric shapes and are constructed of reinforced concrete walls, floors, and roofs. Therefore, based on the configuration and properties of the UHSRS complex, method 2 of ASCE/SEI 7-05 is an appropriate method of wind load design. RCOL2_03.03.02-3

- The exposed portions of the ESWPT (seismic category I) and power source fuel storage vaults (PSFSVs) (seismic category I) are analyzed using method 1 of ASCE/SEI 7-05 (Reference 3.3-1) and an importance factor of 1.15.
- Portions of chases, which are exposed at the interface with other structures, are analyzed using the same ASCE/SEI 7-05 (Reference 3.3-1) method as the structure with which they interface, and an importance factor of 1.15. RCOL2_03.03.02-4 RCOL2_03.08.04-63

CPNPP Units 3 and 4 do not have site-specific seismic category II buildings and structures.

3.3.2.2.2 Tornado Atmospheric Forces

CP COL 3.3(5) Replace the last paragraph in DCD Subsection 3.3.2.2.2 with the following.

Site-specific seismic category I structures include chases, the UHSRS, ESWPT, and the PSFSVs. RCOL2_03.03.02-4 RCOL2_03.08.04-63

The UHSRS, including the pump houses and transfer pump rooms, are configured with large openings and/or vents. The UHS basins and cooling tower enclosures ~~and~~ are ~~therefore~~ designed as vented with respect to tornado atmospheric differential pressure loading. Venting of the pump houses and transfer pump rooms is anticipated during a tornado event, however, for the purpose of structural design, the external walls, internal walls, and slabs of the pump houses and transfer pumps rooms are conservatively designed as unvented and the full tornado atmospheric differential pressure loading is applied. Since the full pressure differential for the structural elements is considered, a depressurization model is not used for the structural design.~~Where applicable, interior walls are designed considering tornado differential atmospheric pressure loading.~~ RCOL2_03.03.02-6

The ESWPT and PSFSV structures are designed as unvented because they do not have openings that permit depressurization during a tornado.

The tornado atmospheric pressure loading on chases is described as follows. The portions of the chases (hollow) which are wholly embedded are not subjected to atmospheric differential pressure effects. The exposed at ground level portions RCOL2_03.03.02-4 RCOL2_03.08.04-63

of chases, which are unvented, are assessed for the full effects of tornado atmospheric differential pressure. RCOL2_03.03.02-4

3.3.2.2.4 Combined Tornado Effects

CP COL 3.3(2) Replace the first and second sentences of the last paragraph in DCD Subsection 3.3.2.2.4 with the following.

Site-specific seismic category I structures, i.e., the UHSRS and exposed portions of chases the ESWPT and PSFSVs, are designed for the same tornado loadings and combined tornado effects using the same methods for qualification described for standard plant SSCs. RCOL2_03.03.02-4 RCOL2_03.08.04-63

3.3.2.3 Effect of Failure of Structures or Components Not Designed for Tornado Loads

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CP COL 3.3(3) Replace the last paragraph of DCD Subsection 3.3.2.3 with the following.

Other miscellaneous NS buildings and structures in the plant yard are located and/or anchored such that their failure will neither jeopardize safety-related SSCs nor generate missiles not bounded by those discussed in Subsection 3.5.1.4. Further, any site-specific or field routed safety-related SSCs in the plant yard are evaluated prior to their installation to determine if structural reinforcement and/or missile barriers are required to ensure their function and integrity.

3.3.3 Combined License Information

Replace the content of DCD Subsection 3.3.3 with the following.

CP COL 3.3(1) ***3.3(1)*** *Wind speed requirements*

This COL item is addressed in Subsection 3.3.1.1.

CP COL 3.3(2) ***3.3(2)*** *Tornado loadings and combined tornado effects*

This COL item is addressed in Subsection 3.3.2.2.4.

CP COL 3.3(3) ***3.3(3)*** *Structures not designed for tornado loads*

This COL item is addressed in Subsection 3.3.2.3.

CP COL 3.3(4) ***3.3(4)*** *Wind load design methods and importance factors*

FIRS4 = nominal response spectrum corresponding to typical plant grade elevation 822' for shallow-embedment structures founded on engineered and compacted structural backfill that extends down to top of limestone at nominal elevation 782'. FIRS4 is computed using both a 30 percent and a 50 percent coefficient of variation for the engineered fill properties to account for a wide range of potential backfill materials. FIRS4 applies to seismic category I duct banks and chases used for routing yard piping and conduits.

The 5 percent damping site-specific horizontal response spectra accelerations for all frequencies, at all FIRS locations, are less than those of the 5 percent damping minimum response spectra tied to the shape of the CSDRS and anchored at 0.1 g, as demonstrated in Figure 3.7-201. Similarly, the 5 percent damping site-specific vertical response spectra, which are developed from the horizontal response spectra using vertical/horizontal response spectral ratios appropriate for the site, are less than the 5 percent damping minimum vertical response spectra tied to the shape of the CSDRS and anchored at 0.1g. The nominal site-specific response spectra described above are less than the minimum required response spectra, and are therefore not used for site-specific design. Instead, the site-specific SSE and FIRS are defined as the shape of the CSDRS anchored at 0.1g, in order to comply with the intent of Appendix S (IV)(a)(1)(i) of 10 CFR 50 (Reference 3.7-7). The site-specific SSE, defined at ground surface (plant grade elevation 822 ft), consistent with the requirements of Appendix S, is the same as the FIRS used as input for site-specific seismic design. By definition, the site-specific SSE and FIRS are automatically enveloped by the CSDRS given in DCD Figures 3.7.1-1 and 3.7.1-2 for standard plant seismic category I structures. The site-specific FIRS (CSDRS anchored at 0.1 g) are used for the design of seismic category I and II SSCs that are not part of the US-APWR standard plant. RCOL2_03.08.04-64 RCOL2_03.08.04-64

The site-specific SSE and FIRS are presented in Figures 3.7-202 and 3.7-203 for the horizontal and vertical ~~FIRS~~directions, respectively. Tabulated values of the corresponding spectral accelerations for each of the spectral control points are presented in Tables 3.7-201 and 3.7-202 for the horizontal and vertical ~~FIRS~~directions, respectively. RCOL2_03.08.04-64 RCOL2_03.08.04.64

CP COL 3.7(2) Replace the seventeenth paragraph in DCD Subsection 3.7.1.1 with the following.

The site-specific verification analysis of US-APWR standard plant seismic category I structures has been performed considering SSI effects and using the site-specific FIRS as described in Subsection 3.7.2.4.1.

CP COL 3.7(13) Replace the first and second sentences of the nineteenth paragraph in DCD Subsection 3.7.1.1 with the following.

For CPNPP Units 3 and 4, the value of the operating-basis earthquake (OBE) ground motion that serves as the basis for defining the criteria for shutdown of the

400 ft. of subgrade below seismic category I and II buildings and structures is 3,830 ft/s. This is above the limit of 3,500 ft/s (corresponding to subgrade material defined as rock with strain-independent dynamic properties) typically used as the cut-off point, below which dynamic testing of the subgrade material would be implemented. At depths below the 400 ft. range discussed above, the shear wave velocity of the rock is higher than 5,500 ft/s. Due to the low site seismicity, the anticipated strains in the rock subgrade due to the site-specific earthquake are very low, less than 0.01 percent. As previously mentioned in Subsection 3.7.2.4, the seismic design of the R/B-PCCV-containment internal structure does not rely on the backfill present on the sides of the building to derive lateral or structural support. Furthermore, the seismic designs of all other seismic category I and II buildings and structures, including the PS/Bs, A/B, T/B, UHSRS, ESWPT, and PSFSVs, also do not rely on backfill for lateral or structural support. The designs of the exterior walls of the building basements consider the earth pressures generated by the design earthquake.

Seismic category I ~~shallow-embedded duct banks and~~ chases are installed in and rest on compacted engineered structural backfill at the site. These structures consist of ruggedly designed reinforced concrete and are equipped with expansion joints that accommodate potentially large strains in the surrounding backfill. RCOL2_03.08.04-63

Based on these site conditions, in which the basemats of all seismic category I and II buildings rest directly on limestone or fill concrete, dynamic testing is not required to evaluate the strain-dependent properties of the rock subgrade and compacted backfill at CPNPP Units 3 and 4.

The water table at the site is located below the basemat bottom elevations and is taken as no higher than elevation 780 ft. for purposes of seismic analysis. The P-wave velocities of the saturated rock layers exceed the P-wave velocity of the water (5,000 ft/s). Therefore, the water table elevation does not affect the P-wave velocities of the submerged subgrade materials. Significant variations in the water table elevation and significant variations of the subgrade properties in the horizontal direction are addressed by using additional sets of site profiles.

In order to accurately capture effects of basemat embedment and flexibility, a 3-D finite element model is used to represent the stiffness and mass inertia of the basement in the SASSI model developed for the site-specific SSI verification analysis. To assure proper comparability with the US-APWR standard plant design, the above-ground portion of the R/B-PCCV-containment internal structure is modeled using lumped mass stick models with properties identical to those of the verified and validated lumped mass stick models of the building superstructure used in the US-APWR standard design.

The properties of the SASSI (Reference 3.7-17) seismic model are verified by an SSI analysis of the building resting on the surface of a hard rock subgrade that simulates fixed base conditions. The results of the SASSI analysis are

Table 3.7-201

Site-Specific Horizontal SSE and FIRS Acceleration Values and Control Points(1), (2), (3)

RCOL2_03.08.04-64

CP COL 3.7(5)

Control Point (Hz)		Acceleration (g)
0.5 percent Damping		
A	(50)	0.1
B	(12)	0.50
C	(2.5)	0.60
D	(0.25)	0.073
E	(0.1)	0.012
2 percent Damping		
A	(50)	0.1
B	(12)	0.353
C	(2.5)	0.43
D	(0.25)	0.057
E	(0.1)	0.009
5 percent Damping		
A	(50)	0.1
B	(12)	0.26
C	(2.5)	0.313
D	(0.25)	0.047
E	(0.1)	0.008
7 percent Damping		
A	(50)	0.1
B	(12)	0.23
C	(2.5)	0.273
D	(0.25)	0.043
E	(0.1)	0.007
10 percent Damping		
A	(50)	0.1
B	(12)	0.19
C	(2.5)	0.23
D	(0.25)	0.04
E	(0.1)	0.006

RCOL2_03.08.04-72 RCOL2_03.08.04-72 RCOL2_03.08.04-72 RCOL2_03.08.04-72 RCOL2_03.08.04-72

Notes:
1) 0.1 g PGA
2) Amplification factors are based on RG 1.60, Rev. 1 (Reference 3.7-6).
3) For Control Points D and E, acceleration is computed as follows: RCOL2_03.08.04-72

Acceleration= $(\varpi^2 D / 386.4 \text{ in/sec}^2) \times F_A \times 0.1$

$\varpi = 2\pi \times$ frequency (rad/sec)

D = Displacement (in)

F_A = Amplification Factor from RG 1.60

Table 3.7-202

Site-Specific Vertical SSE and FIRS Acceleration Values and Control Points(1), (2), (3)

RCOL2_03.08.04-64

CP COL 3.7(5)

Control Point (Hz)		Acceleration (g)
0.5 percent Damping		
A	(50)	0.1
B	(12)	0.50
C	(3.5)	0.57
D	(0.25)	0.05
E	(0.1)	0.008
2 percent Damping		
A	(50)	0.1
B	(12)	0.353
C	(3.5)	0.407
D	(0.25)	0.04
E	(0.1)	0.006
5 percent Damping		
A	(50)	0.1
B	(12)	0.26
C	(3.5)	0.30
D	(0.25)	0.031
E	(0.1)	0.005
7 percent Damping		
A	(50)	0.1
B	(12)	0.23
C	(3.5)	0.26
D	(0.25)	0.029
E	(0.1)	0.005
10 percent Damping		
A	(50)	0.1
B	(12)	0.19
C	(3.5)	0.217
D	(0.25)	0.027
E	(0.1)	0.004

RCOL2_03.08.04-72 (E, 0.008)
RCOL2_03.08.04-72 (E, 0.006)
RCOL2_03.08.04-72 (E, 0.005)
RCOL2_03.08.04-72 (E, 0.005)
RCOL2_03.08.04-72 (E, 0.004)

Notes:
1) 0.1 g PGA
2) Amplification factors are based on RG 1.60, Rev. 1 (Reference 3.7-6).
3) For Control Points D and E, acceleration is computed as follows: RCOL2_03.08.04-72

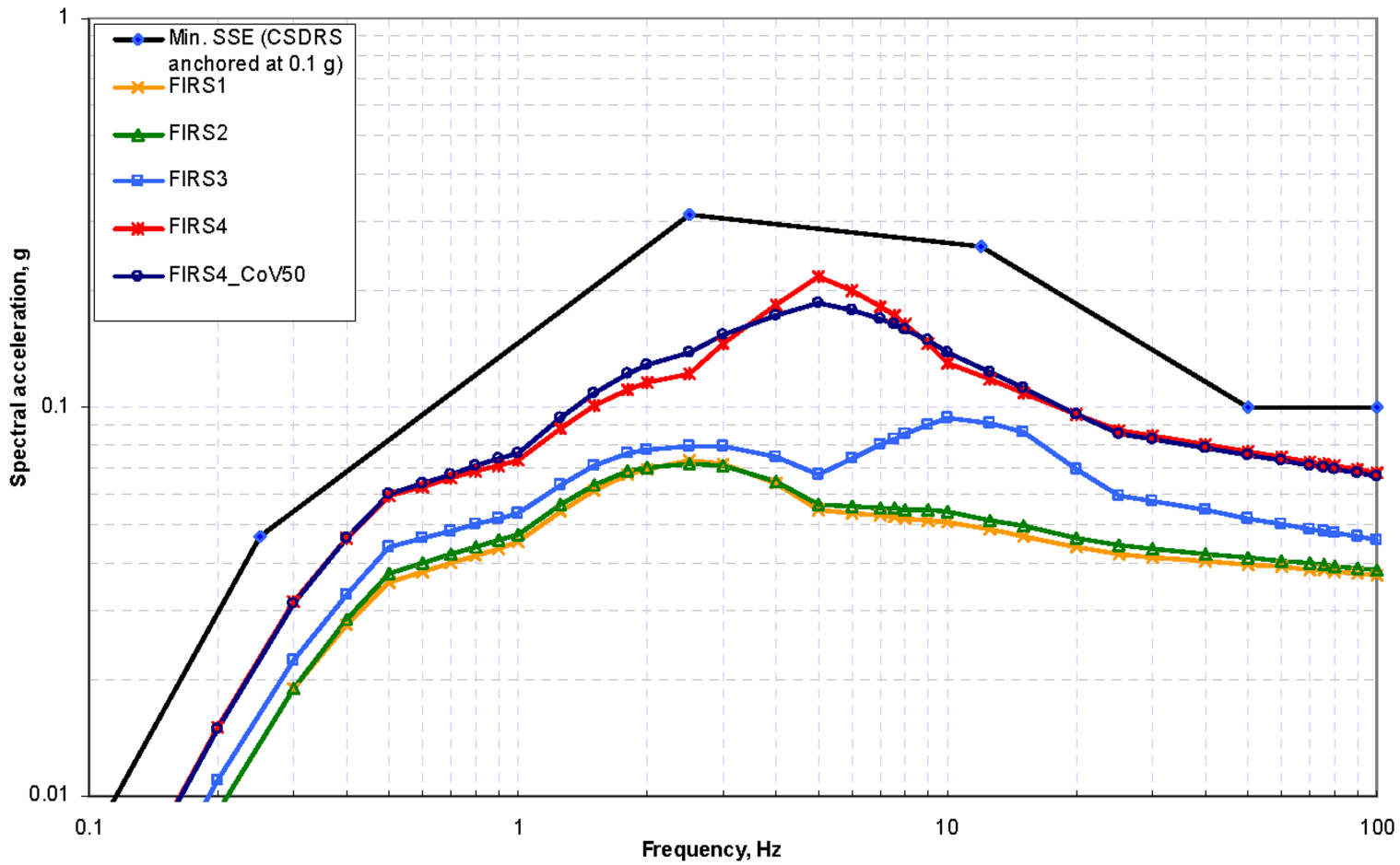
Acceleration= $(\varpi^2 D / 386.4 \text{ in/sec}^2) \times F_A \times 0.1$

$\varpi = 2\pi$ x frequency (rad/sec)

D = Displacement (in)

F_A = Amplification Factor from RG 1.60

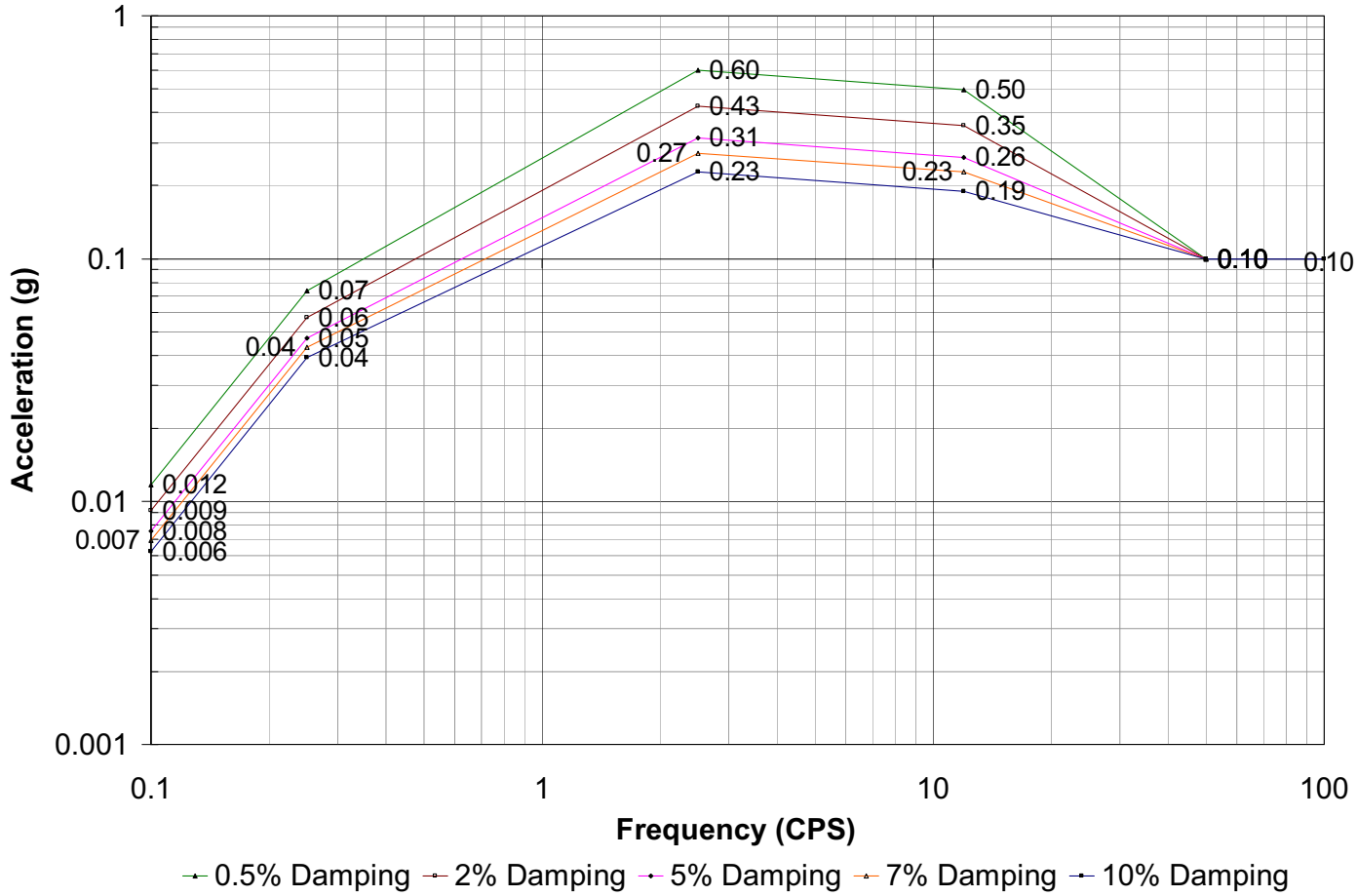
RCOL2_03.08.04-72



CP COL 3.7(5)
CP COL 3.7(6)

Figure 3.7-201 Nominal Horizontal GMRS and FIRS(1),(2) (Sheet 1 of 2)

RCOL2_03.08.04-72

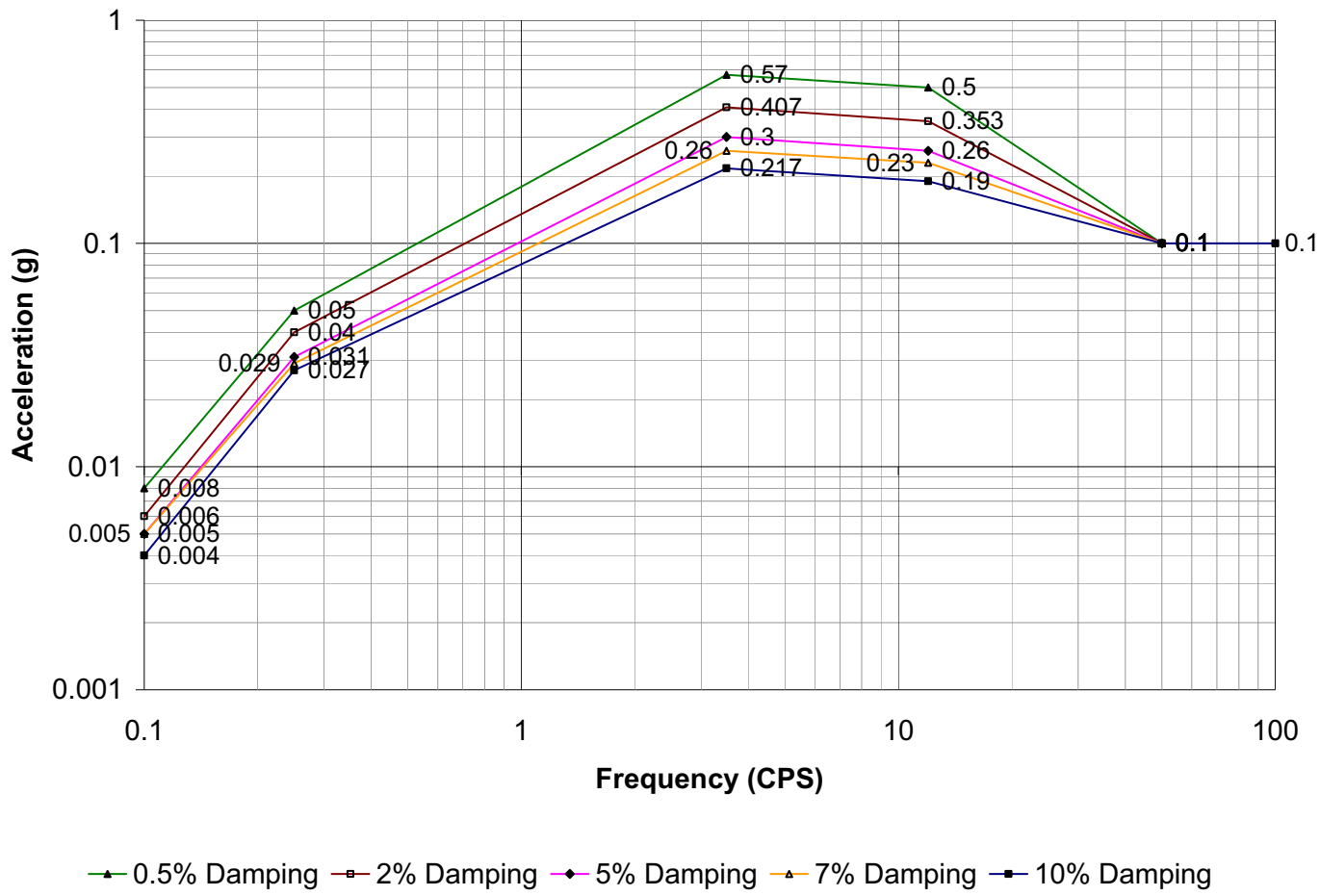


CP COL 3.7(5)

Figure 3.7-202 Comanche Peak Site-Specific Horizontal SSE and FIRS

RCOL2_03.08.04-64

RCOL2_03.08.04-72



CP COL 3.7(5)

Figure 3.7-203 Comanche Peak Site-Specific Vertical SSE and FIRS

RCOL2_03.08.04-64

- Detensioning tendons and the removal of a wire or strand for inspection for corrosion and testing to measure strength and elongation.
- Establish acceptability and compare measured lift-off values with predictions and minimum requirements.
- General visual inspection of all accessible concrete surface areas to assess the general structural condition of the containment.

3.8.4 Other Seismic Category I Structures

CP COL 3.8(15) Replace the fourth paragraph in DCD Subsection 3.8.4 with the following.

The ESWPT, UHSRS, and PSFSVs are site-specific seismic category I structures. These structures are discussed in detail in Subsection 3.8.4.1.3. No site-specific seismic category II structures are applicable at CPNPP.

3.8.4.1.3 ESWPT, UHSRS, PSFSVs, and Other Site-Specific Structures

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CP COL 3.8(19) Replace the second paragraph in DCD Subsection 3.8.4.1.3 with the following.

The ESWPT, UHSRS, and PSFSVs are designed to the site-specific SSE, and are described in detail in Subsections 3.8.4.1.3.1, 3.8.4.1.3.2, and 3.8.4.1.3.3, respectively. Figure 3.8-201 provides the general arrangement of ESWPT, UHSRS, and PSFSVs. Each of these structures is separated from other structures with expansion/isolation joints as shown in various views in Figures 3.8-201 through 3.8-214. The performance specifications for the elastomeric joint or seal materials address requirements for critical characteristics such as bounding the allowable stress-strain properties, durability requirements, and associated material testing. In lieu of expansion joints, the interfaces below grade may be left empty and waterproof joint sealants provided along the perimeter at grade. The sealant will be inspected periodically to maintain integrity. RCOL2_03.08.04-2 RCOL2_03.08.04-61

3.8.4.1.3.1 ESWPT

The ESWPT is an underground reinforced concrete structure. Figure 3.8-203 shows the typical section of the ESWPT. The tunnel layout is a rectangular configuration forming a closed looped structure starting at the UHS Basins and terminating at the T/B. The outside dimensions of the tunnel are shown in Figure 3.8-203. The tunnel is divided into two sections by an interior concrete wall to provide separation of piping trains. Each section contains both ESWS supply and return lines. End walls are also provided where required to maintain train

structure. In this segment there are unbalanced soil pressures, thus requiring shear keys to resist the lateral forces. Higher bearing pressures are placed on the mat slab as well due to overturning moments and a greater overall weight of this segment versus the other segments. RCOL2_03.08.04-1

It is intended that at the interface of two different segments, the interior wall, mat, and slab surfaces line up evenly with the adjacent segments and any difference in slab thicknesses affects only the outer dimensions of the ESWPT segments.

3.8.4.1.3.2 UHSRS

The UHSRS consists of a cooling tower enclosure; UHS ESW pump house, and UHS basin. All of them are reinforced concrete structures, described below.

UHS Basin - There are four basins for each unit and each reinforced concrete basin has one cooling tower with two cells. Each basin rests on a separate foundation, is square in shape, constructed of reinforced concrete, and separated from the adjacent basin by a minimum 4 inch expansion joint. A site-specific specification for the expansion/separation joint that provides material or system performance requirements will be prepared. Performance requirements for an elastomeric material include requirements bounding the allowable stress-strain properties, durability requirements, and specification for a material testing program. See Section 3.8.4.1.3 for alternate to expansion joints. Each basin serves as a reservoir for the ESWS. There is a cementitious membrane adhered to the interior faces of the reinforced concrete walls of the basins which minimizes long-term seepage of water from the basin. An UHS ESW pump house is located at the south-west corner of each basin. Adjacent to the pump house on the east side of the basin are cooling tower enclosures supported by UHS basin walls. The ESWPT runs east-west along the south exterior wall of the UHS basin, and is separated by a minimum 4 inch expansion joint. RCOL2_03.08.04-2 RCOL2_03.08.04-61 RCOL2_09.02.05-3

Each basin is divided into two parts, as shown on Figure 3.8-206. The larger section of the basin shares the pump house and one cooling tower cell enclosure. The other cooling tower cell enclosure is in the smaller segment of the basin. A reinforced concrete wall, running east-west, separates the cooling tower enclosure basin area from rest of the basin. This wall is provided with slots to maintain the continuity of the reservoir.

See Figure 3.8-206 for general arrangement, layout, and dimensions of the UHSRS.

UHS ESW pump house - The pump house is an integral part of the UHS basin supported by UHS basin exterior and interior walls. Each pump house contains one ESW pump and one UHS transfer pump with associated auxiliaries. The pump bay (lowest portion of the pump house required for the pump suction) is deeper than the rest of the UHS basin. A reinforced concrete wall, running east-west, divides the pump house basin from rest of the UHS basin. This wall is provided with slots for flow of water. Two baffle walls (running east-west) are provided inside the pump house basin, before the pump bay. These baffle walls

prevent full penetration or structural failure by the spectrum of tornado missiles identified in Subsection 3.5.1.4. RCOL2_09.02.05-3

For details see Figures 3.8-207 through 3.8-211 for the UHS basin, UHS ESW pump house and cooling tower enclosures. Details of the UHSRS seismic analysis are provided in Appendix 3KK. RCOL2_03.08.04-12

3.8.4.1.3.3 PSFSVs

The PSFSVs are underground reinforced concrete structures required to house the safety-related and non safety-related fuel oil tanks. There is one vault for each PS/B. The vault contains two safety-related and one non safety-related oil tanks. Each tank is contained in a separate compartment. Compartments are separated by reinforced concrete walls. A common mat supports the tanks and the rest of the vault. The PSFSV roof slab is sloped to facilitate drainage. The highest point of the roof slab is slightly above grade. Bollards and a concrete curb are provided to prevent vehicular traffic on the roof.

Access to each vault is provided by a reinforced concrete tunnel from the applicable PS/B. Each tank compartment has a separate pipe/access tunnel, which is an integral part of the ESWPT.

For vault details see Figures 3.8-212 through 3.8-214. Details of the PSFSV seismic analysis are provided in Appendix 3MM. RCOL2_03.08.04-4

3.8.4.1.3.4 Other Site-Specific Structures

There are no additional seismic category I structures.~~Site-specific seismic category I yard piping and conduits are routed within reinforced concrete duct banks (solid) or reinforced concrete chases (hollow). The duct banks and chases have shallow embedments and are buried partially or wholly below grade within structurally engineered and compacted backfill that extends down to top of limestone at nominal elevation 782 ft. The duct banks and pipe chases are constructed in segments, which are separated from each other and other structures by expansion joints.~~ RCOL2_03.08.04-63 RCOL2_03.08.04-5 RCOL2_03.08.04-5 RCOL2_03.08.04-5

~~The expansion joints accommodate all anticipated differential settlement and movement (due to seismic and other loading) at support points, penetrations, and entry points into other structures.~~ RCOL2_03.08.04-5

3.8.4.3 Loads and Load Combinations

CP COL 3.8(20) Replace the second paragraph in DCD Subsection 3.8.4.3 with the following.

Externally generated loads from the following postulated site-specific sources are evaluated in the following subsections:

- Subsection 2.4.2.3 concludes no loads induced by floods are applicable.

3.8.4.4.3.1 ESWPT

The ESWPT is designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the ESWPT is performed using the computer program ANSYS (Reference 3.8-14). The seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3LL.

The static analyses are performed on the ANSYS model placed on soil springs at the top of the concrete fill representing the stiffness of the support provided by the concrete fill and limestone. The stiffness of the subgrade springs under different sections of the ESWPT is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34), for vibration of a rectangular foundation resting on an elastic half space. The springs are included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil stiffness adjacent to the tunnel is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based upon. ~~Since the support below the structure (fill concrete and rock) will not exhibit long-term settlement effects, t~~The subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The same stiffness is applied to all springs and the sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98. In the vertical direction, the smaller of the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures.

RCOL2_03.08.04-8
RCOL2_03.08.04-65
RCOL2_03.08.04-15
RCOL2_03.08.04-69
RCOL2_03.08.04-15

Gravity loads on the tunnel roof include a design surcharge pressure and are resisted by one-way slab action of the roof. These loads are distributed to the outer and interior walls, transferred through the walls down to the mat slab where they are distributed, and from the bottom of the mat slab to the concrete fill over limestone bedrock. A design surcharge pressure of 600 psf is applied to tunnel segments 1 and 2 and a design surcharge pressure of 200 psf is applied to tunnel segment 3.

RCOL2_03.08.04-7
RCOL2_03.08.04-7

Lateral soil pressures on outer tunnel walls are typically resisted by one-way action of the outer walls. Forces from these pressures are transferred to the roof and mat slabs. Where axial force in the roof and mat slabs transverse to the tunnel axis are not balanced by an equal and opposite force from the other side of the tunnel, the roof and mat slabs work with the walls as a moment frame to resist the unbalanced lateral forces. ~~Some~~Corner tunnel segments resist unbalanced

RCOL2_03.08.04-9
RCOL2_03.08.04-9

lateral loads in part by moment frame action and in part by return walls located at an end of the segment (such as where the ESWPT changes direction).

Lateral forces that are not balanced by an equal and opposite force on the other side of the tunnel are transferred to the concrete fill below the tunnel by friction, and where a shear key is present, by friction and lateral bearing of the shear key on the fill concrete. Lateral forces in the fill are then transferred to bedrock by friction, and where required, by lateral bearing of another shear key that extends into bedrock.

For dynamic forces oriented parallel to the length of the tunnel segment, the roof slab acts as a diaphragm that transfers loads to the outer and interior walls. The walls act as shear walls that transfer the forces to the mat slab. For dynamic forces acting perpendicular to the length of the tunnel, the roof acts as a frame member that transfers loads to the interior and exterior walls. The tunnel walls, roof, and base slab act as a moment frame causing out-of-plane bending in these elements. The exterior walls are also designed for static and dynamic soil pressure. The static soil pressures are calculated using at-rest pressures with K_o = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from the overburden pressure and the soil compaction pressure. The dynamic soil pressures are described in Appendix 3LL. ~~in accordance with ASCE 4-98 (Reference 3.8-34).~~ RCOL2_03.08.04-11 RCOL2_03.08.04-11

3.8.4.4.3.2 UHSRS

The UHSRS are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the UHSRS is performed using the computer program ANSYS (Reference 3.8-14). The seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3KK.

The seismic responses for the design are calculated using a multi-step analysis method as defined in ASCE 4-98 (Reference 3.8-34). Step 1 is the SSI analysis using the program SASSI and step 2 is calculating the seismic demands for the design using the program ANSYS as described below. RCOL2_03.08.04-12 RCOL2_03.08.04-68

The ANSYS design analysis models for the UHSRS were placed on soil springs calculated by methods provided in ASCE 4-98 (Reference 3.8-34) to provide localized flexibility at the base of the structure. The flexibility of the base allows for calculation of the base slab demands. The effects of embedment are included in the SSI analysis. The seismic lateral pressure and inertia loads applied to the ANSYS design model represent the total seismic loading from the SSI analysis.

ANSYS analyses are performed based on two support conditions: (1) flexible rock subgrade by applying soil springs across all base slab nodes and (2) rigid base by applying fixed restraints across all base slab nodes. All results from these two conditions are enveloped for design. ~~on the model placed on soil springs at the bottom of the base slab, with the springs representing the stiffness of the rock subgrade. To address the sensitivity of the structural response on the subgrade~~ RCOL2_03.08.04-19 RCOL2_03.08.04-32

~~stiffness, an additional set of analyses simulating a fixed base condition is performed on the model.~~ The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs were included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil adjacent to the UHSRS is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 393 ft. Since the support below the structure will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads.

RCOL2_03.08.04-32
RCOL2_03.08.04-12
RCOL2_03.08.04-32

The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The same stiffness is applied to all springs and the sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98 (Reference 3.8-34). In the vertical direction, the smaller of the spring stiffness that matches the ASCE 4-98 vertical or rocking stiffness is used. Matching of the torsional stiffness is not considered since significant torsional response is not expected (or observed) in any of the structures.

RCOL2_03.08.04-15
RCOL2_03.08.04-69
RCOL2_03.08.04-15

Each UHS cooling tower, air intake enclosures, and ESWS pump house are designed for tornado wind and tornado generated missiles and in-plane and out-of-plane seismic forces. The walls are shear/bearing walls carrying the loads from the superstructure and transferring to the basemat. The UHS basin exterior walls are also designed for static and dynamic soil pressure, and hydrostatic and hydrodynamic fluid pressures. The static soil pressures are calculated using at-rest pressures with K_o = 0.47. This is the same as the at-rest pressure coefficient given in Figure 2.5.4-243. The design also considers the load from soil compaction pressure. The dynamic soil pressures are determined in accordance with ASCE 4-98 (Reference 3.8-34) and the hydrodynamic fluid pressures are determined using ACI 350.3-06 (Reference 3KK-5) and modeling procedures of ASCE 4-98 as described in Appendix 3KK. Below-grade walls loaded laterally by soil pressure on the outside, or hydrostatic pressure on the inside, act as two-way slabs, spanning horizontally to perpendicular shear walls, and cantilevering vertically from the mat slab (at the pump room, the walls span vertically between the mat slab and the pump room floor). For seismic loads, the shear walls are designed to resist 100% of the applied lateral load through in-plane shear. The shear walls transmit load to the mat slab. The shear in the mat slab is transferred to the fill concrete via friction, and direct bearing at the pump house sump. The shear in the fill concrete is transferred to the bedrock via friction and bearing at the

RCOL2_03.08.04-16
RCOL2_03.08.04-13

pump hose sump. The coefficients of friction considered at the fill concrete/bedrock interface and the foundation concrete/fill concrete interface are no higher than 0.6, which is consistent with the values for coefficient of friction discussed in Subsection 2.5.4.10.5. RCOL2_03.08.04-10

Above grade walls loaded laterally by seismic forces as described in Appendix 3KK, or by wind or tornado wind, atmospheric and missile loads, act as two-way slabs, spanning horizontally to perpendicular shear walls and vertically to floor and roof slabs. These slabs act as horizontal diaphragms, and span horizontally to the perpendicular shear walls. The shear in the shear walls is transferred to bedrock as described above. RCOL2_03.08.04-13

Vertical loads in the floor and roof slabs are due to dead load, live load, and wind or tornado missile loads. The floor and roof slabs act as two-way slabs, spanning to the walls or beams below in both directions. The vertical loads are transmitted to the mat slab, then into the fill concrete, and then into bedrock.

3.8.4.4.3.3 PSFSVs

The PSFSVs are designed to withstand the loads specified in Subsection 3.8.4.3. The structural design of the PSFSV is performed using the computer program ANSYS (Reference 3.8-14). Details of ~~T~~the seismic analysis and the computer programs used for the seismic analysis are addressed in Appendix 3MM. RCOL2_03.08.04-4

The ANSYS analyses are performed on the model placed on soil springs at the bottom of the concrete fill / top of limestone level representing the stiffness provided by the rock subgrade. The stiffness of the subgrade springs is calculated using the methodology in ASCE-4 Section 3.3.4.2 (Reference 3.8-34) for vibration of a rectangular foundation resting on an elastic half space. The springs are included to provide localized flexibility at the base of the structure to calculate base slab demands. The soil adjacent to the PSFSVs is not included in the design model in order to transfer the total seismic load through the structure down to the base slab. Embedment effects are included in the SSI model from which the seismic lateral soil pressures and inertia loads are based. The evaluation of subgrade stiffness considers the best estimate properties of the layers above elevation 215 ft. Since the support below the structure will not exhibit long-term settlement effects, the subgrade stiffness calculated from ASCE-4 Section 3.3.4.2 is used for analysis of both static and seismic loads. RCOL2_03.08.04-14 RCOL2_03.08.04-14

The equivalent shear modulus for the ASCE spring calculations is based on the equivalent shear wave velocity which is determined using the equivalent shear wave travel time method described in Appendix 3NN. The equivalent Poisson's ratio and density are based on the weighted average with respect to layer thickness. The springs are included in the model using three individual, uncoupled uni-directional spring elements that are attached to each node of the base mat. The same stiffness is applied to all springs and the sum of all nodal springs in each of the three orthogonal directions are equal to the corresponding generalized structure-foundation stiffness in the same direction calculated from ASCE 4-98 (Reference 3.8-34). In the vertical direction, the smaller of the spring RCOL2_03.08.04-15 RCOL2_03.08.04-69

A comparison of the SASSI generated site-specific in-structure response spectra at the base slab to the ANSYS input spectra confirms that the input used for the ANSYS analyses is conservative. A comparison of the SASSI generated soil pressures with the soil pressures used for the seismic soil pressure analyses performed in ANSYS confirms that the applied loading used for design exceeds that calculated in the SASSI analyses. RCOL2_03.07.02-16 CTS-00922

The seismic design forces and moments resulting from the design analysis are presented in Table 3KK-5 at key UHSRS locations. The force and moment values represent the enveloped results for the seismic demands for all soil cases considered in the SASSI analyses. The seismic demands calculated using the ANSYS design model have been confirmed to exceed the demands calculated by the SASSI analysis. RCOL2_03.08.04-66

Table 3KK-6 summarizes the resulting maximum displacements for enveloped seismic loading conditions at key UHSRS locations obtained from the seismic analysis.

3KK.4 In-Structure Response Spectra (ISRS)

The enveloped broadened in-structure response spectra (ISRS) calculated in SASSI are presented in Figure 3KK-3 for the UHSRS base slab, pump room elevated slab, pump room roof slab, and cooling tower fan support slab for each of the three orthogonal directions (east-west, north-south, vertical) for 0.5 percent, 2 percent, 3 percent, 4 percent, 5 percent, 7 percent, 10 percent and 20 percent damping. The ISRS for each orthogonal direction are resultant spectra, which have been combined using SRSS to account for cross-directional coupling effects in accordance with RG 1.122 (Reference 3KK-7). The ISRS include the envelope of the ~~6~~six site conditions (BE, LB, UB, and HB, ~~with and~~ BE without backfill separation from the structure, and the no-fill surface foundation condition with LB subgrade conditions). All results have been broadened by 15 percent and all valleys removed. ~~It is permitted to perform 15 percent peak clipping of the spectra presented herein in accordance with ASCE 4 (Reference 3KK-3) for spectra with less than 10 percent damping.~~ For the design of seismic category I and II subsystems and components mounted to the UHSRS walls and slab, it is required to account for the effects of out-of-plane ~~wall~~ flexibility, including seismic anchor moments. RCOL2_03.07.02-15 RCOL2_03.07.02-15 RCOL2_03.07.02-15 RCOL2_03.08.04-33

3KK.5 References

3KK-1 *An Advanced Computational Software for 3D Dynamic Analysis Including Soil Structure Interaction,* ACS SASSI Version 2.2, Ghiocel Predictive Technologies, Inc., July 23, 2007.

3KK-2 ANSYS Release 11.0, SAS IP, Inc. 2007.

3KK-3 *Seismic Analysis of Safety-Related Nuclear Structures,* American Society of Civil Engineers, ASCE 4-98, Reston, Virginia, 2000.

RCOL2_03.07.02-16

Table 3KK-9

Comparison of Major Structural Modes of UHSRS between ANSYS Design Model and SASSI SSI Model(1)

RCOL2_03.08.04-70

Mode	Frequency (Hz)		Modal Participation Factor (calculated per ASCE 4-98)(4)		Modal Mass Ratio	
	ANSYS Design Model(2)	SSI Model Mesh(3)	ANSYS Design Model(2)	SSI Model Mesh(3)	ANSYS Design Model(2)	SSI Model Mesh(3)
E-W, Mode 1	6.77	7.08	7.07	7.28	0.251	0.306
E-W, Mode 2	6.55	6.78	2.93	2.48	0.043	0.035
E-W, Mode 3	4.15	4.48	2.89	2.84	0.042	0.047
N-S, Mode 1	7.37	7.62	5.86	5.84	0.172	0.203
N-S, Mode 2	11.49	11.23	2.44	3.55	0.030	0.075
N-S, Mode 3	13.86	14.73	2.33	2.38	0.027	0.033
Vertical, Mode 1	17.37	17.73	2.15	2.00	0.023	0.020
Vertical, Mode 2	10.65	10.67	2.05	1.91	0.021	0.018
Vertical, Mode 3	12.88	16.89	2.04	1.90	0.021	0.018

1. All eigenvalue analyses are performed in ANSYS

RCOL2_03.08.04-18

2. ANSYS Design Model is the fine mesh model used to calculate demands for design
3. SSI Model Mesh is the identical mesh of the UHSRS used for SSI analysis but eigenvalue analysis is performed in ANSYS
4. Modal Participation factors reported are based on total model mass. Active mass is 87% and 50% of the total mass for horizontal and vertical directions respectively.

RCOL2_03.08.04-70

3LL MODEL PROPERTIES AND SEISMIC ANALYSIS RESULTS FOR ESWPT

3LL.1 Introduction

This Appendix discusses the seismic analysis of the essential service water pipe tunnel (ESWPT). The computer program SASSI (Reference 3LL-1) serves as the platform for the soil-structure interaction (SSI) analyses. The three-dimensional (3D) finite element (FE) models used in SASSI are condensed from FE models with finer mesh patterns initially developed using the ANSYS computer program (Reference 3LL-2). The dynamic analysis of the SASSI 3D FE model in the frequency domain provides results for the ESWPT seismic response that include SSI effects. The SASSI model results for maximum accelerations, ~~and~~ seismic soil pressures and base response spectra are used as input to the ANSYS models for performing the detailed structural design, including loads and load combinations in accordance with the requirements of Section 3.8. Table 3LL-14 summarizes the analyses performed for calculating seismic demands. The SASSI analysis and results presented in this Appendix include site-specific SSI effects such as the layering of the subgrade, flexibility, and embedment of the ESWPT structure, and scattering of the input control design motion. Due to the low seismic response at the Comanche Peak Nuclear Power Plant site and the lack of high-frequency exceedances, the SASSI capability to consider incoherence of the input control motion is not implemented in the ~~design~~analysis of the ESWPT. Wave passage effects are considered small and do not impact the seismic design because the tunnel foundation is supported by a stiff limestone layer, which will experience low strains under the fairly low seismic motion at the site.

RCOL2_03.07.02-16
RCOL2_03.07.02-16
RCOL2_03.08.04-35
RCOL2_03.08.04-78

3LL.2 Model Description and Analysis Approach

The ESWPT is modeled with three separate models, each model representing a physical portion of the ESWPT, which are separated by expansion joints (see Subsection 3.8.1.6) that prevent any significant interaction of segments at the interface. Tunnel Segment 1 represents a typical straight north-south tunnel segment buried in backfill soil. Tunnel Segment 2 represents east-west segments adjacent to the ultimate heat sink related structures (UHSRS). Two tornado missile shields extend from the top of this segment to protect the essential service water (ESW) piping and openings into the ultimate heat sink (UHS). The FE model for Segment 3 represents east-west segments adjacent to the power source fuel storage vault (PSFSV) and includes elements representing the fuel pipe access tunnels that extend across the top of the ESWPT. The SSI analyses for all tunnel segments considered soil on all sides in which soil is in contact including the top and bottom of the tunnel.

RCOL2_03.08.04-36
RCOL2_03.07.02-16
RCOL2_03.08.04-40

The ~~FE~~SSI models for each of the three ESWPT segments are shown in Figures 3LL-1 through 3LL-6 as overall and cutaway views. Tables 3LL-1, 3LL-2, and 3LL-3 present the properties assigned to the structural components of the SASSI FE models for Segments 1, 2, and 3, respectively. Detailed descriptions and figures of the ESWPT including actual dimensions are contained in Section 3.8. Shell elements model the roof, interior, and exterior walls, and basemat. Brick

Transfer functions are examined for each analysis to verify that the interpolation was reasonable and that the expected structural responses are observed. Transfer functions, spectra, accelerations, and soil pressures are compared between the various soil profiles used in analyses to verify that the responses were reasonably similar between these cases except for the expected trends due to soil frequency changes. RCOL2_03.07.02-16

Operating-basis earthquake (OBE) structural damping values of Chapter 3 Table 3.7.1-3(b), such as 4 percent damping for reinforced concrete, are used in the site-specific SASSI analysis. This is consistent with the requirements of Section 1.2 of RG 1.61 (Reference 3MM-4) for structures on sites with low seismic responses where the analyses consider a relatively narrow range of site-specific subgrade conditions.

The SASSI analyses produce results including peak accelerations, in-structure response spectra, and seismic soil pressures. All results from SSI analyses represent the envelope of the nine soil conditions. The SASSI analysis results are used to produce the final response spectra and provide confirmation of the ANSYS design input and output demands. RCOL2_03.07.02-11

ANSYS analyses are used to calculate the structural demands of the PSFSV to seismic soil pressure and seismic inertia which are then added to the effects of all other design loads discussed in Section 3.8.4.3. Seismic inertia is analyzed in ANSYS by applying equivalent static lateral loads. The equivalent static lateral loads applied are based on the enveloped peak accelerations calculated in SASSI (provided in Table 3MM-5 and discussed in the following section). For reference, the modal properties of the ANSYS design model are provided in Table 3MM-9. RCOL2_03.08.04-46

The seismic soil pressure is analyzed statically in ANSYS. The seismic soil pressure demands are applied on the structural elements as equivalent static pressures. The pressures applied are shown to be conservative when compared to the calculated elastic solution used in ASCE 4-98 based on J.H. Wood, 1973 and the enveloped SASSI results.

Demands from the equivalent static accelerations and soil pressure analyses performed in ANSYS are combined on an absolute basis to produce the maximum demand in each direction.

The effects of fuel tank flexibility are accounted for in the design of the base slab and global stability criteria. The fuel tank flexibility is accounted for by applying an acceleration on the tanks equal to 1.5 times the peak of the 2% damped base spectra from the SASSI analysis. RCOL2_03.08.04-62 RCOL2_03.08.04-81

3MM.3 Seismic Analysis Results

Table 3MM-4 presents a summary of SSI effects on the seismic response of the PSFSV. The maximum absolute nodal accelerations obtained from the ~~time-history~~SASSI analyses of the PSFSV models are presented in Table 3MM-5. The RCOL2_03.07.02-11

Table 3MM-5

SASSI FE Model Component Peak Accelerations

Component	N-S Acceleration (g) (+/- Y Direction)	E-W Acceleration (g) (+/- X Direction)	Vertical (g) (+/- Z Direction)
North Exterior Wall	0.18	0.18	0.13
South Exterior Wall	0.21	0.17	0.13
West Exterior Wall	0.16	0.42	0.13
East Exterior Wall	0.15	0.26	0.13
West Interior Wall	0.17	0.67	0.13
East Interior Wall	0.17	0.67	0.13
Roof Slab	0.17	0.21	0.63
Basemat	0.11	0.12	0.12

Notes:

1) The peak accelerations presented above envelope all of the considered site conditions, i.e. PSFSV embedded in BE, LB, UB, and HB backfill with and without soil separation, as well as the PSFSV supported by a surface foundation.

2) For structural design using the loads and load combinations in Section 3.8, the seismic loads are obtained by applying uniform accelerations to the PSFSV structure. This approach captured effects due to localized peak accelerations presented above. The uniform accelerations are applied as follows: For the horizontal direction a uniform acceleration of 0.25g was applied. For the vertical direction a uniform acceleration of 0.15g was applied. These accelerations were applied to all elements in the vault ~~(including tanks).~~, except that the horizontal acceleration applied to the tanks for purposes of basemat design was 0.87g. This value represents 1.5 times the peak horizontal acceleration of 0.58g obtained from the 2% damping ISRS for the base slab given in Figure 3MM-3 (Sheet 1 of 6). An additional distributed load corresponding to 0.40g was applied to the two interior walls and the east wall to account for increased local out-of-plane accelerations obtained from the SASSI analysis. An additional distributed load corresponding to 0.50g was applied to all the roof slabs to account for increased local accelerations obtained from the SASSI analysis. Seismic load also includes seismic backfill pressures on the sides of the PSFSV walls. RCOL2_03.08.04-62

Table 3MM-6

Maximum Component Seismic Forces and Moments

Component		Maximum component forces and moments							
		N_V (k/ft)	N_L (k/ft)	Q_V (k/ft)	Q_L (k/ft)	S_W (k/ft)	M_V (k-ft/ft)	M_L (k-ft/ft)	M_{VL} (k-ft/ft)
South Exterior Wall	+	65.07	54.87	14.32	23.61	41.24	25.70	28.28	13.42
	-	87.05	63.09	10.58	24.39	24.18	39.11	68.79	14.45
North Exterior Wall	+	22.62	6.88	4.06	2.02	29.98	9.37	27.50	3.60
	-	19.94	15.12	19.53	3.54	19.54	12.38	15.04	4.68
West Exterior Wall	+	20.07	17.25	19.82	5.27	19.90	76.89	26.73	29.56
	-	15.06	27.82	14.26	13.00	14.06	119.32	48.10	30.14
East Exterior Wall	+	13.82	24.29	6.40	4.71	16.40	34.89	32.23	7.53
	-	16.42	17.29	6.28	5.52	14.10	37.00	14.21	8.06
West Interior Wall	+	25.13	4.29	9.18	5.27	18.51	18.97	11.95	3.38
	-	17.33	31.42	5.31	4.95	13.27	19.53	12.14	3.28
East Interior Wall	+	12.04	4.14	5.20	9.63	17.96	18.75	14.01	3.92
	-	12.87	32.65	6.50	7.75	8.89	19.75	16.26	3.56
Roof Slab	+	25.64	20.19	9.78	6.72	21.22	19.77	8.82	6.74
	-	43.10	20.47	10.99	7.73	17.65	21.19	20.59	7.06
Basemat	+	~~13.71~~ 30.98	~~19.23~~ 29.30	~~18.68~~ 33.41	~~25.70~~ 40.91	~~21.67~~ 35.66	~~176.90~~ 229.94	~~154.34~~ 238.11	~~58.57~~ 103.25
	-	~~21.55~~ 38.83	~~19.61~~ 29.15	~~18.42~~ 24.73	~~26.43~~ 37.91	~~21.07~~ 34.94	~~84.34~~ 137.87	~~157.24~~ 240.56	~~59.04~~ 102.25

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Notes:

1) The forces and moments shown above include forces and moments due to seismic soil pressure that envelope the all four subgrade ~~site~~shear wave velocity conditions (LB, BE, UB, and HB~~) and any effects due to soil separation.~~ The forces and moments are used for structural design as described in Section 3.8.

RCOL2_03.07.02-14

2) The forces and moments are obtained by combination of the three orthogonal directions used in the model by the Newmark 100%-40%-40% method.

3) In the table above the vertical and longitudinal directions define the plane of the walls. N stands for axial force, Q for out-of-plane shear, S_W for in-plane shear and M for moment. The M_V results in normal stresses in the vertical direction of the wall and similarly, M_L results in normal stresses in the longitudinal (horizontal) direction of the wall, and M_{VL} is the torsional moment on the wall. The Q_V is out-of-plane shear force acting on horizontal cross section of the wall, and Q_L is out-of-plane shear force acting on a vertical cross section of the wall. For the roof slab and base slab the vertical axis is oriented along the east-west direction and the longitudinal in the north-south direction