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

CP COL 1.9(2)-1	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. Values of the horizontal 10^{-5} UHRS and FIRS are shown in Table 2.5.2-229 for the seven spectral frequencies.~~The DCD spectrum envelopes all FIRS down to frequencies of 0.5 Hz. Values of the horizontal 10-4 mean UHRS, 10-5 mean UHRS (both at GMRS/FIRS1/FIRS2 control elevations), and GMRS/FIRS1/FIRS2 are in Table 2.5.2-220 for seven spectral frequencies. Values for remaining FIRS are in Table 2.5.2-222.	CTS-01521 CTS-01532
CP COL 1.9(2)-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.	CTS-01532
	Potential for surface tectonic deformation at site	None within the exclusion area boundary	No potential tectonic surface deformation has been identified at the site.	
CP COL 2.5(1)	Subsurface stability – minimum allowable static bearing capacity	15,000 lb/ft^2	The minimum allowable bearing capacity of the foundation bearing stratum meets or exceeds the DCD requirement	
	Subsurface stability – minimum allowable dynamic bearing capacity, normal conditions plus SSE	~~60~~35,000 lb/ft^2	The minimum allowable dynamic bearing capacity of the foundation bearing stratum meets or exceeds the DCD requirement	RCOL2_02.05.04-17
	Minimum factors of safety for bearing capacity without justification[16]	FS = 2.5 - for static bearing capacity FS = 2.0 - for dynamic bearing capacity	The minimum factor of safety for bearing capacity meets or exceeds the DCD requirement	RCOL2_02.05.04-17

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

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 – liquefaction potential	None (for seismic category I structures)	The site strata is not prone to liquefaction
Minimum angle of internal friction for engineered fill and natural in-situ granular soil subgrades	35°	This requirement is only applicable to the condition where the foundation basemats are underlain by engineered fill and natural in-situ granular subgrades as indicated in COL3.8(35) and DCD Subsection 3.8.5.5.2. Category I and II foundation basemats are only underlain by Limestone Layer C or fill concrete and hence this requirement is not applicable.
Presence of fine-grained materials, i.e., silts and clays classified as ML, CL, MH, CH in the Unified Soil Classification System, within 6 in. of bottom of R/B Complex and T/B basemat	Not Permitted.	The R/B Complex and T/B basemat are underlain by Limestone Layer C or fill concrete. No fine-grained materials are within 6 in. of the bottom of either structure.

RCOL2_02.05.04-17

Table 2.0-1R (Sheet 12 of 14)
Key Site Parameters

	Settlement	Total settlement of R/B complex foundation during construction and operational life(14)(15) ~~6~~9.0 in. Differential settlement across R/B complex foundation in any direction during construction and operational life (14)(15) ~~2.0~~5.5 in. Maximum differential settlement between buildings during operation life(14)(15~~6~~) 0.5 in. Maximum tilt of R/B complex foundation generated during operational life of the plant(14)(15~~6~~) 1/2000	Maximum ~~and differential~~ settlement of all the seismic Category I buildings and structures including R/B Complex, PS/B, ESWPT, UHSRS and PSFSV is less than 0.6 inches, while maximum differential settlement is less than 1/2 in.		RCOL2_02.05.04-17 RCOL2_02.05.04-17 RCOL2_02.05.04-17
CP COL 2.3(3)	Atmospheric dispersion factors (χ/Q values) for Technical Support Center (TSC) HVAC intake for specified release points(2):				
	Plant Vent(5) 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.4×10^{-3} s/m^3 8.0×10^{-4} s/m^3 5.1×10^{-4} s/m^3 3.3×10^{-4} s/m^3	0-2 hrs 0-8 hrs 8-24 hrs 1-4 days 4-30 days	1.1×10^{-3} s/m^3 6.9×10^{-4} s/m^3 2.8×10^{-4} s/m^3 2.1×10^{-4} s/m^3 1.3×10^{-4} s/m^3	
CP COL 2.3(3)	Ground-level containment releases(4) 0-8 hrs 8-24 hrs 1-4 days 4-30 days	 1.9×10^{-3} s/m^3 1.1×10^{-3} s/m^3 7.2×10^{-4} s/m^3 4.8×10^{-4} s/m^3	0-2 hrs 0-8 hrs 8-24 hr 1-4 days 4-30 days	8.0×10^{-4} s/m^3 5.1×10^{-4} s/m^3 2.3×10^{-4} s/m^3 1.6×10^{-4} s/m^3 1.1×10^{-4} s/m^3	

Room HVAC intake. For all release locations except the main steam line break, the Class 1E Electrical Room HVAC intakes are closer to the release points than the Control Room HVAC intakes.

14. Acceptable parameters for settlement without further evaluation.

15. ~~Settlements occuring during construction and operational life.~~Operational life of the plant is considered 60 years (including possible life extension). RCOL2_02.05.04-17

16. Settlements occuring during operational life only.

Figures 2.5.2-246 through 2.5.2-254 present the maximum shear strain profiles in the upper 500 feet of all the FIRS profiles and for the 10^{-4}, 10^{-5}, and 10^{-6} BB spectra. The maximum logarithmic mean shear strain (over the sixty synthetic profiles) for the FIRS3 site column (full site column for GMRS/FIRS1 and FIRS2), shown in Figures 2.5.2-246 through 2.5.2-248, has a peak value of about 0.06%. For the FIRS3_COV50 site column (full site column for FIRS1_COV50and FIRS2_COV50), shown in Figures 2.5.2-249 through 2.5.2-251, the maximum logarithmic mean shear strain value also has a peak value of 0.06%. For the FIRS4_SCSR site column (full site column for FIRS4), shown in Figures 2.5.2-252 through 2.5.2-254, the maximum logarithmic mean shear strain has a peak value of 0.05%. For all the FIRS profiles, the maximum shear strain occurs within the granular fill material in the upper 40 feet of the soil column. CTS-01521

In addition, Figures 2.5.2-255 through 2.5.2-260 present the comparison of the median amplification factors obtained for all the FIRS(GMRS/FIRS/FIRS2, FIRS1_COV50/FIRS2_COV50, FIRS3, FIRS3_COV50, FIRS4, and FIRS4_SCSR) using the 10^{-4}, 10^{-5}, and 10^{-6} HF and BB rock inputs. The HF rock input median amplification factors are either lower than or equal to the BB rock input median amplification factors in all the cases. There is little high-frequency energy in the soil motion (as indicated by the relatively flat character of the HF spectra above about 3 Hz). This is particularly true for the 10^{-4} results since the most of the hazard at all frequencies comes from distant events, as summarized in Table 2.5.2-213. Additionally, Table 2.5.2-213 indicates that the 10^{-5} and 10^{-6} hazards come from more local, smaller magnitude events. Use of the BB amplification factors for all magnitude-distance combinations yield conservative hazard results at 10^{-5} and 10^{-6}.

2.5.2.6 Ground Motion and Site Response Analysis

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.2.6 with the following.

Four FIRS have been identified for the CPNPP Units 3 and 4 and are calculated for both the Safe Shutdown Earthquake (SSE) and Operating Basis Earthquake (OBE) where OBE=(1/3)SSE. The SSE is the envelope of the GMRS and the minimum earthquake requirements 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.

2.5.2.6.1 Ground Motion Response Spectrum (GMRS)

All category 1 structures as well as the Turbine Building Pedestal will be founded directly on or embedded in a stiff limestone (Layer C) at~~, or slightly above or below~~a targeted average foundation elevation of elevation 782 ft. Thus the GMRS/FIRS1 (referred to hereafter as GMRS) represents the top of stiff limestone (Layer C) at, or slightly above or below, foundation basemat elevation for the following safety-related~~ and seismic Category II~~ structures: CTS-01521 RCOL2_02.05.04-26 CTS-01521

Ultimate Heat Sink Related Structures (UHSRS) ~~structures~~ along the top of the reservoir slope. Engineering analysis for this potential condition is presented in Subsection 2.5.5, and shows an adequate factor of safety. RCOL2_02.05.04-26

Thick, undocumented fill was placed in former topographic swale areas north and east of the CPNPP Units 3 and 4 power block footprints (Figure 2.5.4-212). The fill extends to the margin of SCR, and is in hydraulic communication with the reservoir. As a result, groundwater occurs as a perched condition in the swale fill at higher elevations than encountered in the bedrock surrounding the in-filled swale areas. Fill in the eastern swale area has undergone differential settlement, indicated by ground cracks and depressed areas. Stability analysis of the swale fill areas is included in Subsection 2.5.5 and demonstrates an adequate factor of safety against this failure mode. Isopach contour maps showing the elevation of the top of Glen Rose Formation bedrock (Figure 2.5.4-213), Glen Rose Formation Layer C foundation strata (Figure 2.5.4-214), and material exposed at plant grade of elevation 822 ft (Figure 2.5.4-215) show that the swale fill is largely stripped from the plant areas by site grading. Safety-related plant structures are supported by foundations bearing into the competent Glen Rose Formation Layer C limestone below plant grade. As a result, any swale fill that may remain around the perimeter of the plant site does not affect the stability or performance of plant safety-related facilities.

2.5.4.1.6 Non-Tectonic Surface Deformation and Volcanism

Subsection 2.5.3 discusses potential sources of non-tectonic deformation and regional volcanic conditions. The potential for non-tectonic deformation from regional ground collapse, salt migration, glacial rebound, and volcanic processes is negligible. No evidence of deformation from these mechanisms is documented in the site region (200 mi radius). Pleistocene continental glaciations did not extend southward to the latitude of the site region. Thick continental crust and shallow bedrock occur in the site vicinity (25 mi radius), and layers of sedimentary basins and thick regional sequences of collapsible weak sediments do not occur within the site vicinity. No piercement-type salt domes are located within the site area (5 mi radius), and the nearest salt dome is located about 105 miles to the east. No Tertiary or Quaternary volcanic activity occurs within the site region, and the youngest regional magmatic activity is Mesozoic in age and located about 100 miles south.

Therefore, these geologic processes do not present a hazard to the CPNPP Units 3 and 4.

2.5.4.1.7 Groundwater Conditions/Withdrawal

Subsection 2.5.1 summarizes potential issues related to groundwater withdrawal from aquifers beneath the site. Subsection 2.4.12 discusses site groundwater conditions, aquifers, and local and regional groundwater resources and usage. The primary drainage in the site area is SCR, an artificial impoundment of Squaw Creek. The pool elevation of SCR is 775 ft, approximately 47 ft below postin soil and shallow rock, screen the site for possible large solution features in the bedrock surface, and facilitate extrapolation and integration of borehole data. Figure 2.5.4-202 shows the locations of geophysical surface surveys.

Field boring logs, daily field reports, and other field records were maintained by a rig geologist assigned to each drill rig. Soil materials are classified in conformance with ASTM D2487, Standard Classification of Soils for Engineering Purposes and D2488, Standard Practice for Description and Identification of Soils. Rock materials are classified in conformance with Brown, E.T. (Reference 2.5-419); Deere, D.U. (Reference 2.5-407); and Dunham, R.J. (Reference 2.5-418) suggested methods. All field geologists and engineers were trained under the project Quality Assurance (QA) Program. Data Collection Plans and borehole-specific Work Instructions provide QA control for the exploration locations, depths, techniques, sampling, and data collection (e.g., classification and logging). Senior geologists and engineers reviewed all field data collection activities and performed independent review of classification and logging operations.

Borings for geotechnical purposes were advanced in soil using Hollow Stem Auger (HSA) drilling techniques and equipment until auger refusal was encountered. Standard Penetration Test (SPT) drive samples were typically obtained on 2.5-foot intervals in soil materials, beginning at about 1 ft below ground surface. All SPT samplers consisted of 18-inch-long standard unlined split barrel drive samplers, and were in good condition. Auger stems had ~~a nominal~~an approximate outside diameter of 8 in and an inside diameter of 4.5 in. Drive sampling by SPT method was conducted with automatic trip hammers with a weight of 140 pounds and a drop of 30 in per ASTM D1586. Blow counts were measured for three consecutive 6-inch driving intervals. In zones with high blow counts, driving was terminated at a count of 50 blows in any 6-inch interval and the actual penetration distance was recorded. Blow counts were recorded independently by rig geologists and drillers and immediately noted on the borehole logs. RCOL2_02.05.04-26

After recovery, rig geologists selected representative portions of each SPT sample and placed each in one or more labeled glass jars with sealed, lined caps. All samples were immediately assigned alphanumeric sample identifications, photographed, described, and recorded on field logs. Use of thin-wall, 3-inch inside diameter Shelby tube samplers (ASTM D1587) was also tried for obtaining undisturbed soil samples. However, due to sampler damage caused by the presence of gravel, large-size particles, and very stiff to hard soil conditions, the attempts were not successful.

Field SPT energy measurements were made for each drill rig on select exploratory borings and recorded during sampling at several different intervals. The ratio of average measured energy to the theoretical potential energy of the SPT system is the energy transfer ratio (ETR). The ETR range of automatic hammers used at the CPNPP site ranges between approximately 66 percent and 92 percent of the theoretical potential energy, with an average value of 82 percent. These ETR values are within the range of typical values for automatic

Surface refraction results were checked against data developed by control borings, test pits, and CPT soundings, and used to develop initial 2-D layered velocity models. These velocity models were then evaluated to identify geologic contacts, determine the surface extent of existing fill, and evaluate the top of the rock profile to help identify any possible large dissolution features or shears. In most areas, the seismic refraction surveys were not able to confidently image the bedrock surface. Where possible, the results from the refraction surveys are incorporated in the site engineering stratigraphy and geologic model described in Subsection 2.5.4.3. Detailed descriptions of the seismic refraction survey and results are provided in the Seismic Refraction Survey Report.

2.5.4.2.1.7.2 Suspension P-S Velocity Survey

Borehole Suspension P-S logging was performed in 15 select borings by Geovision Inc., using an OYO Corporation Model 170 commercial probe. The P-S logging equipment obtains discrete P-wave and S-wave seismic velocity measurements in a borehole using a down-hole source, and is a current industry standard method for nuclear site characterization. Seismic velocity profiles developed using the P-S surveys provided the primary data source to characterize the seismic wave transmission characteristics of the site, and to characterize the site according to the US-APWR Key Site Parameters (DCD Table 2.0-1).

P-S logging was performed in select geotechnical HQ boreholes without casing, and using consistent vertical measurement intervals of 1.6 ft. Boreholes were selected to provide complete spatial coverage throughout the CPNPP Units 3 and 4 seismic category I facilities footprints, and general and power block area, as shown on Figure 2.5.4-203 and summarized in Table 2.5.4-207. Surveyed holes included the reactor center points for both of the CPNPP Units 3 and 4, a deep boring midway between the two reactor centerpoints, and distributed around the power block and UHSRS areas. These include the deepest geotechnical core borings that extended to depths of between about 300 ft and 550 ft. RCOL2_02.05.04-26

For the P-S surveys, the down-hole source generated a horizontally propagating impulsive pressure wave in the fluid filling the boring and surrounding the source. This pressure wave was converted to P- and S-waves in the surrounding soil and rock that were detected by two separate receivers at fixed distances above the source. A field check of measured velocity was performed by comparing calculated velocities between the source and receivers, and independently between the two receivers. Acoustic televiewer and caliper surveys, described in following subsections, were used to determine the borehole dimensions and vertical deviation to help verify the quality of the imaged borehole and evaluate possible impacts from deviations in borehole diameter or inclination. Initial depth-velocity plots produced by the P-S surveys were plotted at a common scale and compared against the borehole geologic stratigraphy, RQD/percent recovery, and other collected geophysical data (e.g., gamma) to develop correlations between velocity layers and geologic/rock mass conditions. This process included field review of core samples between project geologists and Geovision, Inc.

personnel, and comparison of velocities measured in discrete shale and limestone marker beds in the Glen Rose Formation.

Excellent agreement was found between velocity layers and geologic stratigraphy (engineering layers presented in Subsection 2.5.4.3), including differentiation of lower-velocity shale intervals as thin as about 1 ft. Individual velocity profiles from successive borings were also indexed by elevation and key marker strata, and found to be very similar across the entire CPNPP Units 3 and 4 site, providing documentation of a high level of lateral uniformity in seismic velocity.

Figures 2.5.4-206, 2.5.4-207, 2.5.4-208, 2.4.5-209, 2.5.4-210, and 2.5.4-211 show correlations between P-S Suspension velocity profiles, the site stratigraphy, and other geophysical and rock mass parameters. Interpreted seismic wave velocity profiles define the vertical variations in P-wave and S-wave velocity through the site geologic stratigraphy. Summary velocities by principal geologic strata are presented in Subsection 2.5.4.4.2.1. An average composite S-wave velocity for the rock mass extending from plant yard grade to a depth of about 530 ft is in the range of about 4000 to 4500 fps. ~~This corresponds to a “firm rock” condition, according to the US-APWR Key Site Parameters (DCD Table 2.0-1).~~ Detailed descriptions of the borehole Suspension P-S logging and results are provided in the Borehole Geophysical Logging Report. RCOL2_02.05.04-26

2.5.4.2.1.7.3 Down-Hole Velocity Survey

Down-hole seismic velocity surveys were performed in the two reactor center point borings, B-1000 and B-2000 (Figure 2.5.4-202), to provide independent borehole measurements (with respect to P-S surveys) of this critical site parameter. The down-hole technique consists of a single borehole geophone that is clamped to the inside of a PVC casing grouted into the borehole after completion of other geophysical techniques that use an uncased hole. The borehole geophone is lowered to the bottom of the casing and progressively raised and set at 2.5 ft to 5 ft vertical intervals for discrete measurements. These surveys were performed by Geovision Inc. using a Geostuff BHG-3, three-component borehole geophone that orients the geophone parallel to the axis of excitation at the surface. This orientation ensures that received signals are of maximum amplitude. The S-wave signals were generated by blows from a sledge hammer weighing approximately 16 lb against the ends of a wooden plank on smooth and level ground, with ends situated equidistant from the hole, and anchored by placing it under the wheels of a truck. The plank is struck alternately on either end and stacked to facilitate identification of the S-wave arrivals. The P-wave signals were generated by vertical blows to a metal plate placed on smoothed and level ground. The P-wave and S-wave velocities were calculated based on measured wave travel, times and distances between the source and receiver for each depth interval.

A Geometrics Geode seismograph is used to collect recorded data from the geophone. Reliable interpretation of the down-hole surface source extended to a depth of about 135 ft in boring B-1000, and 144 ft in boring B-2000.

2.5.4.2.2.2.18 One-Dimensional Consolidation

One-dimensional consolidation tests were performed in accordance with ASTM D2435 Method B on relatively undisturbed core specimens of shale. Consolidation is the process of gradual transfer of an applied pressure from the pore water to the soil structure as pore water is squeezed out of the voids. Low-density or highly porous shale strata may exhibit a potential for consolidation settlement, commonly expressed as a re-consolidation upon applied foundation loading, following some level of stress-relief rebound in the floor of an excavation. For the consolidation test, a laterally confined specimen is subjected to successively increasing vertical pressure, allowing for free drainage from both the top and bottom surfaces. The samples are inundated shortly after application of seating pressure and loads are applied to contain the swelling. A summary of the test sample locations and results is provided in Table 2.5.4-222. The results from the one-dimensional consolidation tests are used to evaluate potential settlement of weak shale beds, discussed in Subsection 2.5.4.10.

2.5.4.2.2.2.19 Rock Specimen Preparation

All rock core specimens were prepared in accordance with ASTM D4543. This standard outlines the procedure and methods for laboratory specimen preparation and determination of the length and diameter of rock core specimens and the conformance of the dimensions with established standards. Because the dimension, shape, and surface tolerances of rock core specimens are important in determining rock properties of intact specimens, great care is exercised when preparing core samples for strength testing. The prepared cores are measured to determine the straightness of the specimen’s cylindrical side, flatness of its ends, parallelism of the end platens, and perpendicularity of end surfaces to the specimen axis.

2.5.4.2.3 Material Properties

As described in Subsection 2.5.4.3, the CPNPP Units 3 and 4 site is underlain by shallow bedrock comprised of the following main geologic formations, in order of increasing depth: the late Cretaceous Glen Rose Formation limestone and shale (engineering Layers A through F) to an approximate elevation of 620 ft; the late Cretaceous Twin Mountain Formation sandstone, shale, and limestone (engineering Layers G through I) between approximately elevations 620 ft and 390 ft; and the Upper Paleozoic Mineral Wells Formation indurated shale and sandstone below elevation 390 ft. Based on the dimensions, loads, and embedment depths of the seismic category I and II structures, the main zone of foundation influence occurs within Glen Rose Formation engineering Layer C, which consists primarily of competent, massive limestone at and below the targeted average foundation subgrade elevation of 782 ft. RCOL2_02.05.04-26

Laboratory testing included multiple samples of each engineering layer and comprised a complete section through the three main geologic formations. Limited test results are also provided for surficial residual soil and localized 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. There are no site-specific seismic category I structures resting on backfill. Layer C has a uniform thickness of about 60 ft and a consistent S-wave velocity of about 6300 fps~~e~~. Subsurface conditions to a depth of about 550 ft are described in the following subsections. RCOL2_02.05.04-26

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 velocity, natural gamma signature). Assigned engineering layers are laterally continuous throughout the CPNPP Units 3 and 4 site (and extend to the Units 1 and 2 site), and exhibit relatively constant thickness and material properties. Little to no lateral variations or changes are observed in the individual engineering layers throughout the site, based on characteristics observed in numerous borings and geophysical surface and borehole surveys.

The engineering layering follows an alpha-numeric system starting with the shallowest Glen Rose Formation upper limestone strata (Layer A) that occurs at, or near, the ground surface (locally buried by residual soil and/or fill). The vertical segregation of the profile into generalized engineering layers is based primarily on lithologic layers that can be correlated from borehole to borehole, and by geophysical survey velocity layers. The Glen Rose Formation is divided into

Contours are shown in 2-foot intervals where data are dense, and intervals of 5 ft where data are less dense.

Each of the contours was drawn using solid lines where data were available and dashed lines where data were inferred. Contours are shown as solid lines where subsurface control is good, and as dashed lines where extrapolated between widely spaced control points (e.g., borings).

The soil isopach (thickness) contour map (Figure 2.5.4-212) shows the distribution and thickness of surficial residual soil and undocumented swale fill. The thicknesses of soil and fill are variable throughout the plant site area, typically ranging between about 5 ft and 15 ft thick for residual soil, and between about 10 ft and 75 ft for swale fill. The swale fill thickness exhibits a steep gradient of increasing thickness near the margin of SCR, and typically ranges between about 5 ft and 15 ft within the CPNPP Units 3 and 4 power block and footprints, and between about 10 ft and 45 ft in the UHSRS areas. As discussed in Subsection 2.5.4.2, extensive subsurface explorations by borings, test pits, CPT soundings, and geophysical surface refraction surveys provide a high degree of control to define the margins of the swale fill areas with respect to the plant power block footprints. Based on information provided in Figure 2.5.4-212, residual soil and swale fill is largely stripped from the power block areas by mass excavation to form the plant grade at elevation 822 ft (Figure 2.5.4-215). Only thin residual soil or fill, about 2 ft to 5 ft, remains along localized areas of the power block perimeters. RCOL2_02.05.04-26

Deeper excavation for power block and seismic category I and II foundations extends into Glen Rose Formation rock far below the remaining residual soil or fill. The thin mantle of residual soil and fill locally exposed in the upper parts of the perimeter foundation excavations is readily removed or laid back to mitigate any potential adverse impacts (e.g., shallow slumping or erosion into excavations). Essentially, the entire height of the foundation excavations along the power block perimeter is made in the Glen Rose Formation engineering Layers A and B.

Foundation excavations for the UHSRS ~~structures~~ encounter relatively thick deposits of swale fill that locally form a large percentage of the height of the excavation walls. These excavation slopes are laid back or supported to provide temporary construction support, as described in Subsection 2.5.4.5, and backfilled after construction. The foundation subgrade for the UHSRS is extended into competent Glen Rose Formation engineering Layer C, removing any fill from under the structure footprint. Geotechnical inspection of the exposed subgrade during site grading verifies that competent bedrock formation is exposed. RCOL2_02.05.04-26 RCOL2_02.05.04-26

Figure 2.5.4-213 shows contours defining the elevation of the top of sound rock, correlative with Glen Rose Formation engineering Layer A throughout the CPNPP Units 3 and 4 plant site. An irregular bedrock surface, developed by past erosion, exhibits an overall slope to the north and east towards SCR. Former topographic swales northeast of Unit 4 and east of Unit 3 were eroded approximately 10 ft to 25 ft into bedrock prior to later in-filling by undocumented fill and residual soil.

Variations in the elevation of the top of rock, about 15 ft to 25 ft, occur within the power block footprints. The top of rock typically occurs above plant grade elevation of 822 ft, resulting in exposure of a flat rock surface at yard grade over most of the power block area (Figure 2.5.4-215). The top of rock elevation is more variable in the UHSRS areas, with differential elevations of about 30 ft to 40 ft (Figure 2.5.4-213). Massive excavation only partly exposes Glen Rose Formation engineering Layer A rock within the UHSRS footprint areas. The top of rock remains below the elevation of plant yard grade under the northeast portions of the Units 3 and 4 UHSRS footprint areas, but is reached by deeper foundation excavations that extend into competent engineering Layer C limestone (Figures 2.5.4-210 and 2.5.4-211). RCOL2_02.05.04-26 RCOL2_02.05.04-26 RCOL2_02.05.04-26

Elevation contours of the top of Glen Rose Formation engineering Layer C, supporting seismic category I and II structures, are shown on Figure 2.5.4-214. The contoured contact is a conformable bedding contact in the Glen Rose Formation that exhibits an overall gentle east to northeast dip of less than about 1 degree, consistent with the regional bedrock structure discussed in Subsection 2.5.1. This contact represents an essentially horizontal buried surface within the restricted power block footprint area. The average elevation of the top of engineering Layer C is approximately 782 ft below the Unit 3 and Unit 4 power block (Figure 2.5.4-214). The Layer C contour map demonstrates the geometry of the foundation interface for plant structures, and shows that the foundation layer satisfies the US-APWR Key Site Parameters (DCD Table 2.0-1) criteria for maximum slopes of foundation bearing stratum of less than 20 degrees from horizontal.

2.5.4.4 Geophysical Surveys

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.4 with the following.

Geophysical surveys included both down-hole and surface surveys using methods described in Subsection 2.5.4.2.1.7. The following subsections describe how each of the techniques were integrated and applied to characterization of the subsurface conditions.

2.5.4.4.1 Integration of Geophysical Data

Subsection 2.5.4.2.1 describes the locations and general methodology for geophysical surveys at the CPNPP Units 3 and 4 plant site. Detailed results from the surface and borehole geophysical surveys are presented in project data reports. Integrated summary results from these surveys are described herein.

Locations, methodologies, and applications for borehole geophysical measurements are discussed in Subsection 2.5.4.2.1.7. The resulting geophysical measurements provide important independent correlation of bedrock stratigraphy and structure, as well as measurements of in situ engineering and seismic wave transmission properties. This information is integrated to develop a 3-dimensional geologic model of the volume of rock under, and within the foundation influence

- Backfill material types, sources, specifications, and quality control observation and testing
- Foundation excavation, subgrade, and slope geologic monitoring during construction

Figures 2.5.4-209, 2.5.4-210, 2.5.4-211, and 2.5.4-217 illustrate the general layout and general excavation requirements for the main plant structures. Figures 2.5.4-246 and 2.5.4-247 provide preliminary excavation plans for CPNPP Units 3 and 4, respectively. Preliminary excavation section profiles along three north-south and four east-west directions are shown on Figures 2.5.4-248 through 2.5.4-254 for Unit 3, and on Figures 2.5.4-255 through 2.5.4-261 for Unit 4. For general grading and site preparation to plant yard grade elevation of 822 ft (Figure 2.5.5-204), excavation cuts of up to about 45 ft are required within the CPNPP Units 3 and 4 site. The general excavation cuts completely strip all surficial soils and the upper weathered zones of the Glen Rose Formation engineering Layer A. For foundation installations of the structures within the power block and UHSRS areas, additional temporary excavations are required to depths of approximately 40 ft to 45 ft below the yard grade elevation of 822 ft. As shown on Figure 2.5.4-217, Glen Rose Formation Layer B, which consists of shale beds, daylights into the temporary excavation sidecuts near the bottom of the excavation, creating. potential low strength beds and interfaces. The shale strata are generally horizontal, a geometry that is favorable for stability. However, shale strata are considerably weaker materials than limestone strata, and may undergo significant softening and pose potential sliding surfaces that undermine the rock masses within the excavation banks. Although the construction experience from CPNPP Units 1 and 2 suggests that vertical cuts are viable, construction precautionary and preventing methods (e.g. rock anchors or angle cut) that are typical procedures in bedded rock formations with potential weak zones provide an acceptable level of construction stability and ensure the safety of personnel and workers during construction. Since all temporary excavations are backfilled with engineered compacted fill, the potentially weak shale beds above the elevation of about 782 ft do not cause any hazard or instability issues to any of the CPNPP Units 3 and 4 seismic category I and II structures. RCOL2_02.05.04-26

2.5.4.5.1 Cut, Fill and Excavation Limits

The limits of general site grading, excavation, and backfill for the power plant are shown on the preliminary grading and drainage plans. Site grading does not produce cut or fill slopes that directly support the seismic category I and II structures, or that are in sufficient proximity to be a potential hazard to seismic category I and II structures. Subsection 2.5.5 discusses slope stability analyses of permanent cut and fill slopes, and relationships to seismic category I and II plant structures. All seismic category I and II structures are supported on deeply embedded foundation mats that bear directly on prepared and cleaned sound rock of Glen Rose Formation limestone of engineering Layer C (Subsection 2.5.4.3).

The pre-construction ground surface grades within the CPNPP Unit 3 vary between approximate elevation 830 ft and 855 ft in the power block area, and between elevation 790 ft and 847 ft in the UHSRS area. For Unit 4, ground surface grades vary between approximate elevation 842 ft and 869 ft in the power block area, and between elevation 820 ft and 842 ft in the UHSRS area. Based on the site grading drawings (Figure 2.5.5-204), the post-construction main plant area for both Units 3 and 4 power blocks and UHSRS areas encompasses a rectangular pad roughly 1,700 ft long (east to west) and 1,100 ft wide (north to south) to form a relatively level plant grade ranging between elevations 820 ft and 822 ft. This requires overall area cuts ranging between 8 ft and 47 ft in the power block areas, and cuts of up to 27 ft in the UHSRS areas. The only area requiring fill is the northeast corner of the pad within the eastern two UHSRS ~~structures~~ of Unit 3, where fill of up to 30 ft is needed. RCOL2_02.05.04-26 RCOL2_02.05.04-26 RCOL2_02.05.04-26 RCOL2_02.05.04-26

As discussed in Subsection 2.5.4.3, mass excavation removes all surficial residual soil and undocumented fill from the power block footprints, and exposes a flat surface comprised primarily of Glen Rose Formation engineering Layer A limestone. Figure 2.5.4-215 illustrates geologic layers exposed at plant yard grade elevation of 822 ft. Some residual soil and undocumented fill remains at plant grade in the areas of the UHSRS, but further excavation for the foundations of these structures strips these materials from the structure footprints. Additional excavations approximately 40 ft to 45 ft below plant yard grade elevation of 822 ft are required under the power block and UHSRS footprints to reach foundation basemat elevation of approximately 782 ft. Within east and northeast portions of Unit 3, and possibly in isolated areas of Unit 4, some additional “overexcavation,” possibly to elevations of low as about 778 ft (Figure 2.5.4-214), is required to reach the target Glen Rose Formation engineering Layer C limestone for foundation support. RCOL2_02.05.04-26 RCOL2_02.05.04-26

A stretch of 15- to 50-foot-high cut slopes is formed along the west and south margins of the power plant main pad. These cut slopes have inclinations ranging between about 2(H):1(V) and 3.5(H):1(V). The closest approach between the toe of the cut slopes and seismic category I or II structures is approximately 150 ft, providing a substantial safe distance back from the cut slopes. Along the northern margin of the general plant area in the vicinity of the UHSRS ~~structures~~, fill is placed on the reservoir slopes to form the outbound edge of the power plant yard. The fill slopes are approximately 25 ft to 30 ft high, and are inclined at approximately 2(H):1(V). Where the toe of the fill would otherwise project into the reservoir north of Unit 3, a 15-foot-high vertical retaining wall is constructed to constrain the fill. Stability analysis in Subsection 2.5.5 includes an evaluation of the slope and retaining wall. As discussed previously, the UHSRS ~~structures~~ bear on sound Glen Rose Formation limestone Layer C reached by deep excavation under the structure footprints. The fill slopes north of the UHSRS ~~structures~~ are used to re-establish ground surface grades on the reservoir side, and do not provide support for the UHSRS foundations or structural walls that are designed to be self-standing. RCOL2_02.05.04-26 RCOL2_02.05.04-26 RCOL2_02.05.04-26 RCOL2_02.05.04-26

2.5.4.5.2 Excavation and Excavation Support

Figures 2.5.4-209, 2.5.4-210, and 2.5.4-211 illustrate the general excavation requirements below plant yard grade to reach Glen Rose Formation limestone Layer C that forms the foundation mat subgrade for all seismic category I and II structures and plant power block at the targeted average elevation of 782 ft or on fill concrete which extends from the foundation bottom to the top of solid limestone at the targeted average elevation of 782 ft. Steep to vertical cuts will be made around the perimeters of the power block and UHSRS areas, and a level, cleaned excavation floor in limestone will be developed for foundation inspection and preparation, as is illustrated on Figure 2.5.4-217. Some localized overexcavation may be required below elevation 782 ft to remove weathered, dilated, or shaley rock zones. Any overexcavation areas are backfilled to foundation subgrade elevation with fill concrete. RCOL2_02.05.04-26

Excavation of residual soil, undocumented fill, and the upper several feet of the weathered zone at the top of the Glen Rose Formation can be accomplished with conventional heavy earth moving equipment, possibly with some areas of ripping.

Photograph records of CPNPP Units 1 and 2 construction show near vertical, 80 ft high sidecuts in the Glen Rose Formation bedrock. The photographs show that excavations were made without the use of rock anchors or any other excavation support, and appeared to be stable.

The sequence of Glen Rose Formation rock exposed in the CPNPP Units 1 and 2 excavations are the same layers that occur within the excavation ranges at Units 3 and 4. The extensive network of exploratory core borings and geophysical surveys performed throughout the Units 3 and 4 plant power blocks and UHSRS areas show that the rock is sub-horizontal, relatively uniform, and generally free of major steeply dipping discontinuities, shears, or dissolution zones. The horizontal to sub-horizontal bedding planes between discrete shale and limestone strata that typically are several inches to several feet in thickness are the primary rock mass feature. These conditions are favorable for excavation stability. As discussed in Subsection 2.5.4.1, rock stresses at the site are low, and significant stress-relief effects (e.g., excavation floor heave, sidewall bulging) are not anticipated. Geologic conditions are favorable for stability, and past construction experience for CPNPP Units 1 and 2 using vertical and unsupported deep rock excavations was positive. However, it is conservatively assumed that vertical tension cracks could develop in the rock mass behind excavation faces. Such tension cracks, combined with low strength shale bedding surfaces that daylight near the base of the excavation cuts, potentially form shallow rock blocks that could topple into the excavation in an unsupported condition. RCOL2_02.05.04-26

Analyses of stability of temporary cut slopes indicate that if deep tension cracks were to develop, the computed short-term static factor of safety (construction period) would be less than the conventionally accepted minimum value of 1.3. Slope stability analyses indicate that adequate factors of safety (equal or greater than 1.3) could be achieved with a 0.25(H):1(V) or flatter rock cut slopes.

Two options are considered to permit safe excavation conditions:

- Vertical cuts with rows of rock anchors placed in a top-down sequence during excavation to prevent development of tension cracks; or
- Reduced slopes excavated at a maximum inclination of 0.25(H):1(V) without rock anchors.

Temporary cut slope inclinations in rock no steeper than 0.25(H):1(V) are expected to minimize the adverse effect of tension cracks, although flatter slopes might be locally recommended during construction quality control evaluation, depending on the actual rock conditions encountered in the field.

Cut slopes 40 ft or greater in height are provided with 10-foot wide flat benches at the mid height of the slope to control drainage from runoff during storm events, to provide a catchment to protect workers from loose rocks or materials dropped into the excavation, and to provide a potential access road if additional scaling of the rock surface or any other slope repairs are necessary.

Soil Excavation: Residual soil and/or undocumented fill overlie bedrock in some areas of the site. Interpreted contours of thickness of residual soil and fill materials are shown on Figure 2.5.4-212. Available data suggest that the maximum thickness of fill of nearly 45 ft occurs in the vicinity of the northeast corner of Unit 4, as well as at the southeast corner of Unit 3. Mass excavation to form plant yard grade largely removes these materials, and only localized and relatively thin remaining residual soil and undocumented fill remain north and east of the power block areas, as shown in Figure 2.5.4-215. The exceptions are the UHSRS areas, where some areas of relatively thick residual soil and undocumented fill remain below plant grade. RCOL2_02.05.04-26

Temporary cut slopes in residual soil or undocumented fill are no steeper than 2(H):1(V). These cut soil slopes may require periodic maintenance and need protection against erosion, and include a minimum 6-foot wide bench at the mid height for cases where slopes exceed 25 ft in height.

Rock Excavation: Foundation excavations below plant yard grade in the power block and parts of the UHSRS areas are mainly within the relatively hard limestone of Glen Rose Formation engineering Layers A and B (Subsection 2.5.4.3), as illustrated in Figures 2.5.4-209, 2.5.4-210, and 2.5.4-211. Some shale beds and shaley zones occur in this rock sequence, primarily within the engineering Layer B in the lower parts of foundation excavations. RCOL2_02.05.04-26

The upper several feet of rock is typically moderately weathered and dilated, but below this zone the rock mass is generally only slightly weathered to fresh and tight. Exploratory borings in the rock mass indicated RQD values average over 90 percent (Figure 2.5.4-240) and P-wave velocities with averages between about 7000 to 9000 fps in shale and limestone of engineering Layers A and B,

- Halting floor excavation about 2 ft short of final elevation if an extended exposure time (e.g. over winter) is anticipated prior to placement of concrete. Final excavation to expose and prepare foundation subgrade on the Glen Rose Formation engineering Layer C commences when equipment and materials are ready for concrete placement.

- Applying shotcrete to the excavated faces in shale beds or shaley intervals of limestone. Shotcrete should be provided with weep holes to prevent water pressure buildup.

- Localized overexcavation and dental concrete in possible zones of blast-damaged rock, closely fractured zones, and unusual deep weathering.

Geologic Mapping of Excavation, Documentation and Monitoring: Geologic mapping is required on a continuous basis during foundation excavation, with mapping consistent with the rock and engineering layer classifications described in Subsection 2.5.4.3. Detailed engineering geologic mapping should be supplemented with photographs, video tapes, and topographic survey of the excavated surfaces and pertinent geologic features exposed. All final excavation cuts and foundation subgrade exposures require final inspection and mapping in order to ensure that all shale and unsuitable materials are removed and competent rock materials are exposed.

2.5.4.5.3 Dewatering

As discussed in Subsections 2.4.12 and 2.5.4.1, permanent groundwater occurs deep in the rock mass below plant grade and foundation subgrade elevations. Groundwater inflows into excavations are not considered to be a significant issue, and no significant dewatering or control measures are required during excavation, or for permanent groundwater control. The groundwater elevation at the site meets US-APWR Key Site Parameter (DCD Table 2.0-1) requirements for maximum groundwater level of 1 ft below plant grade.

Possible temporary (e.g., storm-induced) perched water tables that could develop in thin residual soils or undocumented fill that remain in restricted areas of the Units 3 and 4 power blocks and around the UHSRS should drain quickly and not produce significant volumes or rates of inflow into excavations. The perched water table condition can be controlled by having sumps and pumps installed at key locations in the excavations. RCOL2_02.05.04-26

Other than “perched” water, localized water bearing layers or lenses, no groundwater was encountered in the primary Glen Rose Limestone. Therefore only normal pumping equipment and procedures are required to remove storm runoff and concrete curing water that could enter the open excavations.

During construction of CPNPP Units 1 and 2, only small and localized seeps were reportedly observed in foundation excavations that extended to deeper levels (and lower elevations) than at CPNPP Units 3 and 4.

2.5.4.5.4 Backfill Material

Backfill is required between the foundation excavation sidewalls and lower structural walls of seismic category I and II facilities, the main power block structures, and the UHSRS. The volume of backfill is minimized by using steep or vertical excavation cuts. RCOL2_02.05.04-26

No exclusions are placed on the use of limestone or sandstone derived from the mass grading to develop plant grade or foundation excavations. The total volume of excavation in the Units 3 and 4 power block and UHSRS areas greatly exceeds the volume of required backfill. Shale materials are not acceptable for backfill material in structural areas because of their fine-grained nature, high plasticity, and expansion potential. Testing of limestone and shale samples is discussed in Subsection 2.5.4.2. Dynamic properties assigned to engineered backfill are discussed in Subsection 2.5.4.7.4. The source of backfill to be used adjacent to category I structures will be the limestone and sandstone removed from the excavation and that there will be sufficient quantity of material from the excavation for that purpose. The acceptance criteria, test method, and frequency of verification for fill placement are provided for each fill application in Subsection 2.5.4.5.4.8. Continuous geotechnical engineering observation and inspection of all fill is required to certify and ensure that the fill is properly placed and compacted as discussed in Subsection 2.5.4.5.4.2. RCOL2_02.05.04-26

Clean sand may be used as a select granular backfill material around the buried structure walls. A discussion of the materials for engineered fill is provided in Subsection 2.5.4.5.4.1.1. All seismic category I and II buildings and structures are founded directly on solid limestone or fill concrete (Subsection 3.7.1.3). Recommendations for concrete fill under power block structure foundations are provided in Subsection 2.5.4.5.4.1.2.

Concrete fill may be used as backfill to replace unsuitable rock removed below elevation 782 ft as part of foundation preparations. The concrete fill foundation details are shown on Figure 2.5.4-217.

2.5.4.5.4.1 Material Properties and Sources

2.5.4.5.4.1.1 Fill

All engineered fill materials need to contain no rocks or hard lumps greater than three inches in size, and require to have at least 80 percent of material smaller than 1/2 inch in size. No organic, perishable, spongy, or other improper material such as debris, bricks, cinders, metal, wood, etc. shall be present in the fill. Three types of engineered fill materials are used at the site.

At least one in-place moisture content and one density test are required on every lift of fill, and further placement is not allowed until the required relative compaction has been achieved.

The number of tests is increased if a visual inspection determines that the moisture content is not uniform or if the compacting effort is variable and not considered sufficient to meet the project specification.

Light hand-guided compaction equipment is required for compaction of soils within 5 ft of the below-grade walls or other earth-retaining concrete walls. Heavier compaction equipment can be used at distances greater than 5 ft from the walls. The use of light, hand-guided compaction equipment near the walls avoids applying excessive compaction-induced soil pressure against the wall.

2.5.4.5.4.6 Field Monitoring and Quality Control

This subsection describes methods and procedures used for verification and quality control of the foundation subgrades and materials. Properties of the foundation materials are discussed in Subsection 2.5.4.2.

2.5.4.5.4.6.1 Exposed Subgrades

Quality control is required to verify that competent subgrade and quality foundation materials are exposed prior to placement of fill materials. This applies to foundations supported directly on rock, as well as fill or structural concrete. The quality of rock or fill concrete provides very high safety margins against bearing capacity failure under both static and seismic loading conditions, and allows only ~~nominal~~insignificant minimal settlements to occur. RCOL2_02.05.04-17

The quality control testing requirements for rock and concrete foundation material are discussed below.

The procedure for verification of foundation conditions consists of geologic mapping of the final exposed excavation surface prior to placement of foundation concrete or fill concrete.

Geologic mapping of final exposed rock surfaces beneath Units 3 and 4 and any required extension to reach suitable rock material is periodically carried out at a scale of 1 in equals 5 ft. Areas where further detail is needed for documentation of significant features are also documented on the geologic map in order to ensure that all shale and unsuitable materials are removed and competent rock materials are exposed.

The geologic mapping program includes photographic documentation of exposed surfaces and laboratory testing and documentation of significant features.

2.5.4.6.2 Groundwater Occurrence

According to the preliminary results from monitoring of field piezometers within the Units 3 and 4 area, the piezometric levels range between about elevation 775 ft to 858 ft. However, there are also a number of wells that remain dry. Observed piezometric levels are considered to be localized perched water in the upper zone of the Glen Rose Formation, and could possibly be attributed to surface run-off rather than a true indication of permanent groundwater at the site.

As discussed in Subsections 2.4.12 and 2.5.4.1, permanent groundwater occurs deep in the rock mass below plant grade and foundation subgrade elevations. Groundwater inflows into excavations and are therefore not considered to be a significant issue. No significant dewatering or control measures are required during construction excavations. ~~The site grading detail indicates that the potential maximum confined groundwater level within the engineered fill surrounding the main plant area is not expected to exceed elevation 813.5 ft.~~Based on theoretical maximum precipitation events, the maximum groundwater level calculated around the nuclear island is 794.94 ft. The areas of Units 3 and 4 within the Essential Service Water Pipe Tunnel (ESWPT) is essentially a closed basin. The pipe tunnels enclose this area, with the tops of individual segments of the pipe tunnels ranging from 804 ft to 810 ft. Because this is a closed area, the water level within this area can theoretically reach a maximum of 804 ft; once it has reached this elevation, the water will drain outward across those portions of the pipe tunnel having tops at that elevation. Therefore, the conservative potential maximum groundwater level within the engineered fill is considered to be 804 ft. The groundwater elevation at the site meets US-APWR Key Site Parameter (DCD Table 2.0-1) requirements for maximum groundwater level of 1 ft below plant grade. RCOL2_02.05.04-26

2.5.4.6.3 Construction Dewatering

Groundwater, seepage, or runoff, if encountered in open excavations during construction, is anticipated to be of a relatively low volume and may be handled by sumping and pumping. Sumps may be placed within either Glen Rose limestone or sub-foundation concrete that replaces excavated shale materials.

2.5.4.6.4 Groundwater Impacts on Foundation Stability

Because foundations bear directly on limestone with no indication of active karst conditions, as described in Subsection 2.5.1.2.4, or on sub-foundation concrete (that replaces excavated shale materials), the presence of groundwater is not anticipated to significantly impact foundation stability, bearing capacity, or settlement characteristics.

Groundwater or seepage may impact construction activities if water infiltrates shale and claystone materials on excavated side slopes. Shale is likely to deteriorate in the presence of water as a result of excavation and construction traffic that exposes shale surfaces to slaking. Shale materials require removal from trafficked surfaces.

Shale is present at the base of slopes excavated for construction of Units 3 and 4. The surface of shale exposed within the excavated slope is required to be immediately covered by shotcrete or other suitable materials upon completion of excavation to prevent deterioration of shale through exposure to air and/or water.

To minimize the buildup of hydrostatic pressures, adequate drainage behind retaining walls and at the base of all fill slopes along the SCR banks is required. Impacts on the retaining wall design and performance of the fill slopes along the SCR banks are not significant as long as retaining wall foundation and slope drainage systems perform satisfactorily or the hydrostatic pressure buildup is considered in the design.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.7 with the following.

2.5.4.7.1 Overview

This subsection discusses the response of soil and rock to dynamic loading and collection and evaluation of field and laboratory dynamic measurements in order to develop the dynamic site characteristics for seismic design and earthquake engineering purposes. Information presented in Subsections 2.5.1, 2.5.4.1, 2.5.4.2, and 2.5.4.4 form the basis for the dynamic evaluation described herein. The site dynamic properties are used as input for classification of the site in conformance with US-APWR Key Site Parameters (DCD Table 2.0-1), development of the site GMRS presented in Subsection 2.5.2.6.1, and development of FIRS presented in Subsection 2.5.2.6.2. Site dynamic properties also are used for any required SSI analysis as described in Subsection 2.5.2.6.~~2~~3. RCOL2_02.05.04-26

Requirements in 10 CFR Parts 50 and 100 pertaining to site dynamic characterization include:

- An investigation of the effects of prior earthquakes in site soils and rocks including evidence of paleoearthquake liquefaction;
- Field seismic surveys and presentation of interpreted data to develop bounding seismic S-wave and P-wave velocity profiles; and,
- Dynamic laboratory tests on undisturbed samples of foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysteretic damping properties.

All seismic category I and II structures are founded at the targeted average elevation of 782 ft directly on or embedded in competent and massive Glen Rose Formation Layer C limestone, or thin fill concrete placed over the Layer C RCOL2_02.05.04-26 limestone. The GMRS and primary FIRS 1 profiles applicable for these conditions are equivalent, and developed at the targeted average elevation of 782 ft at the top of Layer C limestone, as described in Subsection 2.5.2.6. Additional FIRS profiles are described in Subsection 2.5.2.6. ~~An additional four FIRS profiles (FIRS 2, FIRS 3, FIRS 4_CoV30, and FIRS 4_CoV50) are for specific conditions that are different than the GMRS/FIRS 1 condition. The remaining FIRS are established at plant grade elevation 822 ft and factor combinations of in-place Glen Rose Formation Layers A and B and granular engineered backfill to facilitate evaluation of shallow-embedded plant facilities. The following subsections dscribe development of the site characteristics used as input for the GMRS and FIRS calculations.~~ RCOL2_02.05.04-26

2.5.4.7.2 Site Earthquake Effects

As discussed in Subsections 2.5.2, 2.5.3, and 2.5.4.1, the CPNPP Units 3 and 4 site is located within a stable continent area with relatively low stress conditions and low historic seismicity. No active structural deformation occurs within the site vicinity (25 mi radius), site area (5 mi radius), and site (0.6 mi radius). No Quaternary faults, liquefaction features, or possible tectonic features have been identified within the site vicinity (25 mi radius) by the U.S. Geological Survey or compilation of local mapping, or were identified by aerial photograph analysis and field reconnaissance within the site area (5 mi radius) for the CPNPP Units 3 and 4 investigations. Site subsurface explorations demonstrate that competent Glen Rose Formation bedrock occurs at shallow depths throughout the plant area. This rock is stable and not subject to earthquake-induced ground failure from liquefaction, lateral spreading, or lurching.

As discussed in Subsection 2.5.2, horizontal peak ground accelerations (PGA) range between 0.046g and 0.077g, although 0.10g is used for seismic design per minimum requirement of Appendix S to 10 CFR Part 50 and US-APWR DCD Subsection 3.7.1.1. No significant adverse ground shaking hazard or seismic slope instability is anticipated at the project site based on the low seismicity and estimated PGA values. RCOL2_02.05.04-26

2.5.4.7.3 Field Seismic Velocity Profile Input

Subsection 2.5.4.4.2 discusses the integrated seismic velocity profile for the site, which consists of a shallow profile extending to the maximum depth of site explorations and geophysical surveys, and a deep profile extending from the base of the shallow profile to hard basement.

Figure 2.5.4-239 shows the shallow integrated profile that extends from the ground surface to the maximum depth of site geophysical surveys of approximately 550 ft (elevation 300 ft). ~~On the basis of field measurements, the CPNPP Units 3 and 4 site is classified as a Firm Rock site, according to the US-APWR Key Site Parameters table (DCD Table 2.0-1).~~ RCOL2_02.05.04-26

Extension of the site seismic velocity profile between the bottom of the shallow profile to hard basement that exhibits an S-wave velocity of >9200 fps is described in Subsection 2.5.4.4.2.2. Hard basement is defined at the top of the Ellenburger limestone at a depth of about 5273 ft below plant grade. Table 2.5.2-~~227~~212 presents a stepped deep velocity profile used as input for the GMRS. RCOL2_02.05.04-26

2.5.4.7.4 Dynamic Soil and Rock Input Parameters for GMRS and FIRS

Table 2.5.2-~~227~~212 presents dynamic properties of site soil and rock materials for development of the GMRS and FIRS. These values are based on field and laboratory measurements described in Subsection 2.5.4.2 and the information provided below. RCOL2_02.05.04-26

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 UHSRS, as shown in Figure 2.5.4-215. RCOL2_02.05.04-26 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~~212). RCOL2_02.05.04-26

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~~212, and consider essentially linear response within the seismic strain ranges. RCOL2_02.05.04-26 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 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~~212 are derived based on standard 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 2.5.2-241~~32~~.

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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 0.046~~5~~g and 0.077g, resulting in a minimum design PGA of 0.1g. RCOL2_02.0 5.04-26

All seismic category I and power block structures associated with Units 3 and 4 are founded on or embedded in 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. RCOL2_02.0 5.04-26

The foundation base mats of all seismic category I and II structures are founded on or embedded in a limestone layer (engineering Layer C). RCOL2_02.0 5.04-26

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 because they consist 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 (Reference 2.5-480).

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. 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.

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.

~~This subsection briefly summarizes the derivation of the site GMRS and Safe Shutdown Earthquake (SSE) that are detailed in Subsection 2.5.2.6.~~Derivation of the site GMRS and safe shutdown earthquake (SSE) is detailed in Subsection 2.5.2.6. RCOL2_02.0 5.04-26

The CPNPP Units 3 and 4 site is in a stable continent area with relatively low regional stress and low regional seismicity, as described in Subsections 2.5.1 and 2.5.2, and summarized in Subsection 2.5.4.1. Design ground motions are also relatively low.

A performance-based, site-specific GMRS was developed in accordance with the methodology provided in RG 1.208. This methodology and the GMRS are provided in Subsection 2.5.2.6. The GMRS satisfies the requirements of

10 CFR 100.23 for development of a site-specific SSE ground motion. ~~The SSE is the envelope of the GMRS and the minimum earthquake requirements of 10 CFR Part 50 Appendix S, based on the shape of the Certified Seismic Design Response Spectra (CSDRS) scaled down to a PGA of 0.1g. The CSDRS for the US-APWR is 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.~~ RCOL2_02.05.04-26

~~As recommended in RG 1.208, the following general steps were undertaken:~~

- ~~Review and update the EPRI (1986) (Reference 2.5-369) seismic source model for the site region (200 mi radius), including updated characterization of the Meers fault, which represents the nearest active seismic source to the site~~
- ~~Update the EPRI (1989) (Reference 2.5-370) ground motion attenuation model using the EPRI (2004) (Reference 2.5-401) ground motion attenuation model~~
- ~~Perform sensitivity studies and an updated Probabilistic Seismic Hazard Analysis (PSHA) to develop rock hazard spectra and define the controlling earthquakes~~
- ~~Derive performance-based GMRS from the updated PSHA at a free field hypothetical outcrop at the top of competent material beneath the site (defined as top of Glen Rose Formation Layer C)~~

~~The resulting GMRS and derivative FIRS are presented in Subsection 2.5.2.6.~~

2.5.4.10 Static Stability

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.10 with the following.

2.5.4.10.1 Bearing Capacity

Seismic category I and II structures for Units 3 and 4 are founded on mat foundations bearing directly on or embedded in sound Glen Rose Formation limestone Layer C (Subsection 2.5.4.3), or concrete fill placed over limestone. Strength and compressibility properties for the Glen Rose Formation materials are discussed in Subsection 2.5.4.2. Extensive core borings and geophysical surveys performed throughout the CPNPP Units 3 and 4 seismic category I and II structure footprints demonstrate that the targeted Glen Rose Formation engineering Layer C limestone is approximately 60 ft thick below foundation subgrade elevation, massive, and highly uniform in characteristics. Average RQD of the limestone below the foundation subgrade is greater than 95 percent (Figure 2.5.4-240), and S-wave and P-wave velocities average over 5500 fps and 11,000 fps, respectively (Figure 2.5.4-239). The rock is horizontally to subhorizontally layered, and no significant voids, shears, or weak zones occur in the Layer C limestone that could form potential bearing sliding surfaces or differential settlement. The targeted RCOL2_02.05.04-26 average foundation subgrade elevation of 782 ft provides deep confinement of the limestone of about 40 ft below plant grade, and no slopes or sloping rock surfaces exist around the Units 3 and 4 power blocks that could result in lateral confinement reduction. RCOL2_02.05.04-26

Ultimate bearing capacity for both Units 3 and 4 seismic category I and II structures was estimated for three potential failure mechanisms of general shear failure, local shear failure, and compressive failure, as presented in the Rock Foundations Manual by the U.S. Army Corps of Engineers (COE, Reference 2.5-420).

The traditional Buisman-Terzaghi bearing capacity expression is used to calculate ultimate bearing capacity for the general shear failure condition, as shown below:

$$q_{ult} = cC_c N_c + 0.5\gamma B C_\gamma N_\gamma + \gamma D N_q$$

$$N_c = 2N_\phi^{1/2}\left(N_\phi + 1\right)$$

$$N_\gamma = N_\phi^{1/2}\left(N_\phi^2 - 1\right)$$

$$N_q = N_\phi^2$$

$$N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right)$$

Where:

q_{ult}	=	Ultimate bearing capacity
γ	=	Effective unit weight (i.e. submerged unit weight if below groundwater table) of rock mass
B	=	Width of foundation
D	=	Depth of foundation
c	=	The cohesion intercept for rock mass
ϕ	=	Angle of internal friction angle for rock mass
C_c	=	Foundation shape correction factor for N_c (see Table 6-1, Reference 2.5-420)
C_γ	=	Foundation shape correction factor for N_γ (see Table 6-1, Reference 2.5-420)

N_c, N_γ, N_q = Bearing capacity factors

Local shear failure is a case where a failure surface starts to develop but does not propagate to the surface. For this mode of failure, depth of embedment contributes little to the total bearing capacity. The expression for the ultimate bearing capacity applicable to localized shear failure is as follows:

$$q_{ult} = cC_cN_c + 0.5\gamma BC_\gamma N_\gamma$$

The parameters are the same as those defined for the general shear failure condition.

Compressive failure is a case characterized by a foundation that is supported on poorly constrained columns of rock, and the failure mode is similar to unconfined compression failure. The expression for the ultimate bearing capacity applicable to compressive failure is as follows:

$$q_{ult} = 2c\tan(45 + \frac{\phi}{2})$$

The parameters are the same as those defined for the general shear failure condition. Assuming ϕ=0, the ultimate bearing capacity for compressive failure is approximated by the unconfined compressive strength of rock mass (q_{ult}=2c).

COE recommends that the initial strength parameters selected for analysis should be based on lower bound estimates because rock masses generally provide generous margins of safety against bearing capacity failure. For a conservative estimation of the bearing capacity using the above procedures, the angle of internal friction is assumed to be zero and the cohesion is assumed to be one-half of the lower bound of the unconfined compression strength values.

Results of the bearing capacity analysis performed for main seismic category I and II structures (Table 2.5.4-228) indicate that the ultimate bearing capacity for foundations bearing in Glen Rose Formation engineering Layer C limestone is governed by the compressive failure mode and is at least 146 ksf. The estimated bearing capacity is compared to minimum bearing capacity values referenced in the US-APWR Key Site Parameters (DCD Table 2.0-1) that are 15 ksf static and ~~60~~35 ksf dynamic. The estimated ultimate bearing capacity for engineering Layer C limestone provide factors of safety against bearing capacity failure of about 10 for static loading and at least ~~1.5~~4.17 for seismic loading. The actual available factors of safety for specific structures (Table 3.8-202) are much higher than these levels and clearly indicate that the Glen Rose Formation engineering Layer C limestone provides adequate bearing capacity for support of the proposed structures. RCOL2_02.05.04-17 RCOL2_02.05.04-17 flexible loaded area is expressed as follows (Reference 2.5-438):

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\frac{m^2+n^2+2}{m^2+n^2+1}+\sin^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\right)\right]$$

$$m = \frac{L}{Z}$$

$$n = \frac{B}{Z}$$

Where:

σ_z	=	Stress increment at a depth z
q	=	Uniform load intensity as surface
B	=	Width of the loaded area
L	=	Length of the loaded area
Z	=	Distance below the loaded area
m,n	=	Ratio of loaded area width or length to depth

The vertical stress induced at other locations than the corner or by more than one foundation can be obtained through the superposition approach.

A summary of the results of the settlement and deformation analyses conducted by the non-layered and layered methods described above for the two BE and LB deformation modulus models are presented in Tables 2.5.4-229 and 2.5.4-230, respectively.

Estimated total settlements for seismic category I and II structures founded on Glen Rose limestone Layer C are estimated to be ~~less than~~on the order of 1/2 in. Estimated differential settlement is not anticipated to exceed about 1/3~~4~~ in across the foundation widths or around the perimeters of the structures. Settlement estimates assume excavation procedures do not affect integrity or compromise the load bearing capacity of limestone to any appreciable degree. RCOL2_02.05.04-17

These estimated settlements are consistent with estimated settlements for foundations of CPNPP Units 1 and 2 supported in similar Glen Rose Formation limestone, as discussed in the FSAR (Reference 2.5-201). They conform to total and differential settlement criteria for the US-APWR Standard Design.

Additional information and details regarding the procedure and results of the settlement calculations are provided in the Settlement and Bearing Capacity report.

During construction and after the completion of Units 3 and 4, a number of settlement points or plates will be established on selected parts of the structures for settlement monitoring purposes during the life of the plant. The existing Maintenance Effectiveness Monitoring Program for Units 1 and 2 will also be adopted to carry on the monitoring program for Units 3 and 4.

2.5.4.10.3 Excavation Rebound Potential

As discussed in Subsection 2.5.4.1, regional stresses in the geologic formations at the CPNPP site are low, and significant stress relief during excavation is not expected. Rebound deformation estimates are carried out using a similar procedure as described in Subsection 2.5.4.10.2. The BE modulus profile was considered more applicable and therefore was used for the rebound estimates. Rebound deformation due to removal of about 40 ft of soil and rock material to the top of Layer C limestone rock is not anticipated to exceed about ~~1/8~~0.15 in. A summary of the rebound estimates for the center points of the main structures is shown in Table 2.5.4-231. Based on these results, the potential for any significant heave or rebound of the foundation rock due to foundation excavation during the construction is considered very low. RCOL2_02.05.04-26

The CPNPP Units 1 and 2 FSAR (Reference 2.5-201) discusses rock stress relief measurements associated with general plant site excavation recorded in two extensometers. A maximum rebound of 0.02 in was measured by the extensometers during deep excavation (approximately 30 ft to 60 ft) into upper Glen Rose Formation strata that are laterally contiguous with the rock strata that will be excavated for the CPNPP Units 3 and 4 plant site and seismic category I and II foundations. No occurrences of high stress or stress-induced instability are described.

Additional information and details regarding the procedure and results of the excavation rebound calculations are provided in the Settlement and Bearing Capacity report.

2.5.4.10.4 Lateral Earth Pressures

Lateral earth pressures acting on below-grade structures and walls are due to the self weight of backfill soils, backfill compaction, hydrostatic, surface (temporary or permanent) loads, and transient (seismic) loads.

Typical examples of a lateral active and at-rest earth pressures for select granular backfill are summarized on Figures 2.5.4-242 and 2.5.4-243, respectively. Lateral earth pressures acting on non-yielding walls (rigid and restrained from displacement and rotation), such as the seismic category I and II structures, are to be calculated for an at-rest condition. Other walls that are capable of yielding (including flexible or walls free to displace or to rotate at the top) are calculated for active conditions. Intermediate cases of lateral earth pressure may exist depending on the degree of rigidity, stiffness, and restraining characteristics of the wall system. Detailed methodology and calculations for lateral earth pressures are provided in Chapter 3.

2.5.4.10.5 Resistance to Lateral Loads

Lateral loads can be resisted by an allowable passive soil pressure acting on the sides of the foundations. In addition, lateral loads may be resisted by friction acting along the side walls and the base of the foundation.

Ultimate passive pressures are calculated for select granular backfill and are summarized on Figure 2.5.4-244. The upper 2 ft of passive resistance should be neglected unless the soil is confined by pavement or slab.

For concrete tightly poured against firm foundation limestone bedrock (at approximate elevation 782 ft), base coefficient of friction of 0.6 is applicable for use between the base of concrete foundation and the limestone bedrock interface, or concrete foundation and concrete fill interface.The coefficient of friction is applied to net buoyant (dead, normal) loads for the portion of the structure that extends below the groundwater table.

The recommended coefficient of sidewall friction at the interface between the sidewall and the backfill soil is 0.35. RCOL2_02.05.04-18

~~All seismic category I and II structures are designed based on friction acting along the base of the foundations and by shear keys (if and where needed) for lateral sliding. No passive pressure or frictional resistance along the sides of the foundations or the below-grade structures are used for resisting lateral loads. Additional details are provided in Subsection 3.8.4.~~

2.5.4.11 Design Criteria

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.4.11 with the following.

Methods used to evaluate bearing capacity, settlement and lateral earth pressures are discussed in Subsection 2.5.4.10. Soil and rock properties used in the analyses are provided in Subsections 2.5.4.2 and 2.5.4.3.

The estimated ultimate bearing capacity suggests that minimum factors of safety against bearing capacity failure are approximately 10 for static loading and ~~2~~7 for seismic loading condition. For all seismic category I and II structures, the foundations are founded in Layer C limestone. The estimated total settlements are ~~generally less than~~on the order of 1/2 in, with differential settlements of up to about 1/34 in. Seismic category I and II structures are expected to experience settlements that are within the acceptable criterion. RCOL2_02.05.04-17 RCOL2_02.05.04-17

Fill concrete material is required to meet the requirements as defined in Subsection 2.5.4.5.4.1.2.

2.5.5 Stability of Slopes

CP COL 2.5(1) Replace the content of DCD Subsection 2.5.5 with the following.

In conformance with Regulatory Guide (RG) 1.206, this subsection provides an evaluation of the static and dynamic stability of all natural and man-made earth and rock slopes that could adversely affect the safety of seismic category I and II structures for CPNPP Units 3 and 4. The slope evaluation incorporates site characterization information described in Subsection 2.5.4, and applies geologic- and geotechnical-based slope stability methodology in current practice for nuclear power projects. In general, all seismic category I and II structures within the nuclear islands are founded on stable and competent Glen Rose Formation limestone Layer C at ~~about~~the targeted average elevation of 782 ft or on fill concrete which extends from the foundation bottom to the top of solid limestone at the targeted average elevation of 782 ft. The design of the ~~Ultimate Heat Sinks (~~UHSRS~~)~~ consists of reinforced concrete structures that are also founded on the Glen Rose Formation limestone Layer C, and does not include any earth embankments for side wall support. Geologic conditions, past slope performance, and slope stability analyses presented in this subsection indicate that a postulated failure of soil, fill, or rock materials above Layer C in any slopes in the vicinity of the plant would not adversely affect the safety or performance of seismic category I and II structures. RCOL2_02.05.04-26

Temporary cuts below plant yard grade are required for construction of safety-related structures. However, all temporary cuts and excavations are backfilled with engineered fill up to plant yard grade level, and do not pose any post-construction or operational slope stability hazard. Temporary construction cut slopes are discussed in Subsection 2.5.4.5.

A map showing the locations of the proposed CPNPP Units 3 and 4 plant facilities, with respect to site setting, is shown on Figure 2.5.4-201. Safety-related seismic category I and II facilities are shown on Figure 2.5.4-216.

As specified in RG 1.206 (pages C.I.2-35 to C.I.2-37), this subsection is organized into the following subsections:

- Slope Characteristics (2.5.5.1)
- Design Criteria and Analyses (2.5.5.2)
- Logs of Borings (2.5.5.3)
- Compacted Fill (2.5.5.4)

Slope stability analyses considered temporary and permanent loading conditions, pre- and post-construction topography (Figure 2.5.5-204), groundwater conditions described in Subsections 2.4.12 and 2.5.4.6, and seismic ground motions described in Subsection 2.5.2.

2.5.5.1 Slope Characteristics

2.5.5.1.1 Locations and Descriptions of Slopes

The CPNPP Units 3 and 4 plant yard area is a large (approximately 1700 ft by 1000 ft) level pad at elevation 822 ft (Figure 2.5.5-204). The pre-construction ground surface grade within the power block area of CPNPP Unit 3 varies in elevation between approximately 830 ft and 855 ft, and the grade within the power block area of the CPNPP Unit 4 varies in elevation from approximately 842 ft to 868 ft. Site grading to prepare the level yard grade involves general cut and excavation ranging between approximately 8 ft and 33 ft for CPNPP Unit 3 and from about 20 ft to 46 ft for CPNPP Unit 4.

The plant grade transitions into gently sloping natural and artificial ground along the west, south, and eastern margins of the pad. No slopes of significant gradient and/or height exist in these areas to present a potential slope stability issue. As shown on Figure 2.5.5-204, a combination of natural and graded slopes descends from the northern margin of the plant yard to SCR along the north margin of the plant site and in the area of the UHSRS. Reservoir pool elevation is 775 ft, and the side slopes rising above reservoir level to plant grade are between 40 ft and 45 ft high. The closest approach of these slopes to the plant power blocks are northeast of CPNPP Unit 3, and north to northwest of CPNPP Unit 4. The pre-construction slopes northeast of CPNPP Unit 3 have an overall maximum inclination of approximately 5(H):1(V), and those north and northwest of CPNPP Unit 4 have an overall maximum inclination of approximately 3(H):1(V). Some localized areas may have slightly steeper inclinations. Portions of the slopes also continue for some distance below the reservoir water level.

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Table 2.5.5-201 provides a summary of the post-construction slopes and their pertinent data such as conditions, types, locations, heights, maximum inclinations, and their distances to seismic category I structures.

2.5.5.1.2 Past Slope Performance

There is no evidence of past significant landsliding within a 0.6 mi radius of the CPNPP Units 3 and 4 site, based on aerial photograph evaluation and field reconnaissance mapping. Intact outcropping strata of Glen Rose Formation bedrock are visible tracing along the topographic contour in the area of the reservoir slope on pre-reservoir and modern aerial photographs. Discrete bedrock strata of the Glen Rose Formation can be correlated with borings along the north margin of the plant site at expected elevations based on projections considering bedding dip (nearly flat). This correlation provides geologic evidence that the bedrock has not been displaced by past landsliding.

Localized surficial erosion and raveling has occurred in undocumented fill and/or native colluvial soils on the reservoir slopes. This is considered a routine/normal maintenance issue involving surficial conditions and does not present a significant slope stability hazard to the CPNPP Units 3 and 4 plant site.

Shale layers in the upper parts of the Glen Rose Formation (within engineering Layers A and B; Subsection 2.5.4.3) daylight in the reservoir slopes, as shown on pre-construction Cross Sections D-D' and E-E' (Figures 2.5.5-202 and 2.5.5-203), above reservoir pool level. The dip of the shale beds is near-horizontal, a geometry that is favorable for slope stability and helps limit the size of potential slope failures. Although significant sliding has not occurred to date or during the geologic history of the slopes, and the bedding dip is generally favorable for stability, the beds represent weaker zones in the rock mass that could act as a potential sliding surface, especially if softened by perched groundwater conditions. Stability analysis in Subsection 2.5.5.2 evaluates the long-term slope stability safety factors for this potential failure mode with respect to the UHSRS ~~structures~~ in proximity to the reservoir slopes. Massive, stable limestone of Glen Rose Formation engineering Layer C daylights in the reservoir slope slightly above the pool elevation. This limestone is resistant to sliding, and constrains the depth and toe locations of possible slope failure. Slope failure in limestone at or below the reservoir pool elevation is not likely to occur. RCOL2_02.05.04-26

Potential sliding along shallow bedrock shale beds in Glen Rose Formation Layers A and B would not affect the stability of power block facilities for CPNPP Units 3 and 4 because these structures are founded on Glen Rose Formation Layer C limestone below the shale layers, and are set back a considerable distance from the reservoir slopes.

Thick, undocumented fill in former topographic swale areas north and east of the CPNPP Units 3 and 4 power block footprints (Figures 2.5.4-212 and 2.5.4-215) extends to the margin of SCR, and forms localized portions of the reservoir slopes. The fill bodies appear to be in hydraulic communication with the reservoir. As a result, groundwater occurs as a perched condition in the swale fill, at higher elevations than encountered in the bedrock surrounding the filled swale areas. Fill in the eastern swale area has undergone differential settlement, indicated by ground surface cracks and depressed areas. Sliding failure of undocumented fill over native soils, bedrock, or along failure planes in the fill are modeled by slope stability analysis in Subsection 2.5.5.2. Stability analyses in Subsection 2.5.5.2 evaluate the stability safety factors for this potential failure mode with respect to the UHSRS ~~structures~~ in proximity to the reservoir slopes and fill areas. RCOL2_02.05.04-26

Potential fill sliding would not affect the power block facilities for CPNPP Units 3 and 4 because these structures are founded on competent limestone below the elevation of fill, and are set back from the fill and reservoir slope areas.

Existing permanent slopes associated with the CPNPP Units 1 and 2 include artificial cuts at the intake and discharge structures on the shore of SCR and road cuts. These slopes are made largely in Glen Rose Formation limestone and shale bedrock, but also include engineered fill slopes. Slope heights are typically on the order of about 5 ft to 25 ft, and are inclined between about 2(H):1(V) to near-vertical. Field observations indicate that the existing slopes are generally stable and have performed well since construction that typically occurred 20 to 30 years ago.

The generalized stratigraphy and assignment of engineering layers adopted for use in the slope stability analyses are based on the site geologic-engineering model/profile presented in Subsection 2.5.4.3.

2.5.5.2 Design Criteria and Analyses

2.5.5.2.1 Analysis Cross Sections

Slope stability analyses were performed for five representative post-construction cross sections along the reservoir margin identified on the basis of grading plan inspection. The selected analysis locations include the maximum slope inclinations and permanent slopes at, or in the vicinity of, the UHSRS ~~structures~~. The five analyzed cross sections are labeled D-D', E-E', F-F', G-G', and H-H', and their locations are shown on Figure 2.5.5-201. The cross sections are described below: RCOL2_02.05.04-26

- Cross Section D-D' (Figure 2.5.5-205): This cross section is oriented roughly in a south-north direction and is located northwest of Unit 4, passing through the western UHSRS unit from plant yard grade into the Squaw Creek Reservoir and through an intervening retaining wall structure. Cross Section D-D' ranges in elevation from 819 ft to 758 ft with a resulting total height difference of approximately 61 ft (44 ft above reservoir pool elevation 775 ft). The retaining wall within this cross section is about 17 feet high and extends from elevation 817 ft to 800 ft. This retaining wall extends further below grade to an elevation of about 780 ft in order to be founded a minimum of 2 ft into competent limestone Layer C. This sloping section of the cross section contains one break in slope at approximately elevation 780 ft. Maximum gradients above and below the slope break are approximately 2(H):1(V) (compacted fill and shale over limestone slope), and 1.25(H):1(V) (limestone slope within Squaw Creek Reservoir), respectively. RCOL2_02.05.04-26

- Cross Section E-E' (Figure 2.5.5-206): This cross section is oriented in a southwest-northeast direction and is located northeast of Unit 3, passing through the eastern UHSRS unit into Squaw Creek Reservoir and through an intervening retaining wall structure. Cross Section E-E' ranges in elevation from 819 ft to 762 ft, with a resulting total height difference of approximately 57 ft (44 ft above reservoir pool elevation 775 ft). The retaining wall within this cross section is about 37 feet high and extends from elevation 817 ft to 780 ft. This retaining wall extends further below grade to an elevation of about 778 ft in order to be founded a minimum of 2 ft into competent limestone Layer C. The sloping section of this cross section has a maximum gradient of approximately 3(H):1(V) (limestone slope within Squaw Creek Reservoir). RCOL2_02.05.04-26

- Cross Section F-F' (Figure 2.5.5-207): This cross section is oriented in a southeast-northwest direction, and passes through undocumented fill between Units 3 and 4 and into Squaw Creek Reservoir. Also included is a representation of the Vehicle Barrier System (VBS). Cross Section F-F' ranges in elevation from 817 ft to 744 ft, with a resulting total height difference of approximately 73 ft (42 ft above reservoir pool elevation 775 ft). This section contains two breaks in slope at approximately elevation 795 ft and elevation 780 ft. Maximum gradients above, between, and below these slope breaks are approximately 2(H):1(V) (compacted fill, undocumented fill, residual soils, and limestone slope), 3(H):1(V) (compacted fill and limestone slope), and 2(H):1(V) (limestone within Squaw Creek Reservoir), respectively.

- Cross Section G-G' (Figure 2.5.5-208): This cross section is oriented roughly in a south-north direction and is located northwest of Unit 4, passing through the area west of the western UHSRS unit into the Squaw Creek Reservoir. Cross Section G-G' ranges in elevation from 817 ft to 757 ft, with a resulting total height difference of approximately 60 ft (42 ft above reservoir pool elevation 775 ft). This section contains two main breaks in slope at approximately elevation 795 ft and elevation 790 ft. Maximum gradients above, between, and below these slope breaks are approximately 2(H):1(V) (compacted fill, shale, and limestone slope), 7(H):1(V) (compacted fill, shale, and limestone slope), and 2(H):1(V) (limestone within Squaw Creek Reservoir), respectively. RCOL2_02.05.04-26

- Cross Section H-H' (Figure 2.5.5-209): This cross section is oriented roughly in an east-west direction and is located northwest of Unit 4, passing through western UHSRS unit into a drainage pond located northwest of Unit 4. Cross Section H-H' ranges in elevation from 819 ft to 799 ft with a resulting total height difference of approximately 20 ft. The maximum gradient of this slope is 3(H):1(V) (compacted fill, shale, and limestone slope). RCOL2_02.05.04-26

The cross sections show the post-construction site grading as interpreted from the site grading plans, the interpreted vertical and lateral extent of the surficial soils, and the depth to various bedrock layers. Based on the site grading plans (Figure 2.5.5-204), engineered compacted fill is placed on the reservoir side of the UHSRS units, as shown on post-construction Cross Sections D-D', E-E', and G-G' (Figures 2.5.5-205, 2.5.5-206, and 2.5.5-208). Along the northeast and northwest boundaries of the site, a retaining wall is used to provide a relatively level pad within the plant area (Figures 2.5.5-205 and 2.5.5-206). In areas where undocumented fill or weak shale materials daylight within the slopes, an engineered buttress consisting of compacted fill founded into limestone Layer C is provided to maintain an acceptable stability performance of the slope (Figures 2.5.5-207, 2.5.5-208, and 2.5.5-209). RCOL2_02.05.04-26

2.5.5.2.2.6 Glen Rose Formation Limestone

Glen Rose Formation limestone typically is massive and well-cemented, and it exhibits brittle hard rock strength properties. The shear strength parameters for limestone were derived from laboratory unconfined compression test results that were modified to account for rock mass properties using published strength correlations initially developed by Hoek and Brown (References 2.5-409 and 2.5-410), and subsequently refined to include rock mass disturbance factors (from blasting and stress relief) by Hoek et al. (Reference 2.5-411). The Hoek-Brown criteria consider the scale effect of potential rock mass failure and the weakening influence of joints and other discontinuities in the rock mass. To develop a range of strength values, each unconfined compression test value was used to develop a Hoek-Brown shear-strength vs. normal-stress curve. The range of rock strength envelopes was used to estimate the limestone shear strength, as shown on Figure 2.5.4-237. The lower-bound Hoek-Brown shear strength envelope curve was selected as a conservative strength model for the in situ limestone rock mass. The lower-bound envelope was then used in the slope stability program to estimate the shear strength as a function of effective normal stress.

2.5.5.2.3 Groundwater

Groundwater within the existing fill is controlled by the water level in the adjacent SCR. According to the USGS, the pool elevation of the SCR is normally about elevation 775 ft, and has historically fluctuated between elevations 773 ft and 778 ft. Filled swale areas northeast of CPNPP Unit 4 and east of Unit 3 extend to the reservoir shoreline. The fill appears to be in hydraulic communication with the reservoir, and a perched groundwater table at, or near, the elevation of the reservoir pool exists in the fill. The subsurface native soils and most of the rocks, especially the Glen Rose Formation, are considered relatively impermeable and watertight. ~~However, monitoring well data from onsite piezometers indicate the presence of some localized water at shallower elevations.~~ Based on the site grades and drainage features the maximum potential groundwater level ~~within the engineered fill surrounding the main plant area~~ is not expected to exceed elevation 804~~13.5~~ ft. Groundwater and hydrogeologic conditions of the site are discussed in detail in Subsections 2.4.12 and 2.5.4.6.2. RCOL2_02.05.04-26

For the purpose of modeling the slope stability, the groundwater table was conservatively assumed to be at elevation 804~~13.5~~ ft, within the engineered fill surrounding the main plant structures with a steady state seepage toward the shores of SCR. RCOL2_02.05.04-26

2.5.5.2.4 Slope Stability Analysis Methodology

The slope stability analyses were performed for static and dynamic (pseudo-static) loading conditions. The latter analysis was performed using both horizontal and vertical seismic coefficients.

Conventional two-dimensional limit-equilibrium analyses were performed considering permanent (long-term) slope stability conditions.

Various methods of analysis, including Janbu and Bishop's (References 2.5-428, 2.5-429, and 2.5-430), were used for initial screening of possible failure surface geometries. Various failure surface shapes were considered, including Rankine-type, random block, and circular surfaces. Refined analyses were performed using Spencer's method (Reference 2.5-431) on targeted failure surfaces identified by the screening analysis. Spencer's method is considered more appropriate as it satisfies both force and moment equilibrium.

Soil and rock materials that exhibit anisotropic shear strength properties are more appropriate to be modeled by assigning Mohr-Coulomb strength parameters with two sets of shear strength parameters: "along" and "across" bedding. For conservatism, only along-bedding shear strength parameters of the shale were used in the stability analysis of permanent slopes (Subsection 2.5.5.2.2.5). This approach was used to model the Glen Rose Formation shale beds. Hoek-Brown criteria for rock-mass shear strength parameters were used to model the massive Glen Rose Formation limestone.

The computer program Slope/W 2007 (Geo-Slope International) was used to perform the slope stability analyses. This program models heterogeneous soil types, soil and rock anisotropy, complex stratigraphic and slip surface geometry, and variable pore water pressure conditions. The program was validated and verified for these analyses.

2.5.5.2.5 Dynamic Slope Stability

A pseudo-static method of analysis was used for stability evaluation of the slopes at the project site. In this method, the effects of seismic loading conditions on the slopes are accounted for through the application of constant horizontal and vertical seismic coefficients to the slope and computation of a pseudo-static factor of safety. With the conservative assumption of vertical-to-horizontal ratio of 1.0 the magnitude of the vertical coefficient is taken equal to the horizontal PGA. Both positive (downward) and negative (upward) vertical coefficients were considered. The orientation resulting in the lower factor of safety is considered the critical condition. If pseudo-static slope stability analyses, in which the horizontal and vertical seismic coefficients are taken equal to the PGA, result in factors of safety greater than 1.1, seismic slope performance is considered acceptable.

Ground motion and site response analyses discussed in Subsection 2.5.2 indicate that the horizontal PGA ~~corresponding to the GMRS and FIRS1 at the CPNPP Units 3 and 4 site is about 0.045g. Horizontal PGA corresponding to the other FIRS are all below 0.07g, as shown on Figures 2.5.2-234 and 2.5.2-239~~ranges between 0.046g and 0.077g. Therefore, the US-APWR DCD minimum PGA of 0.10g is used as the design PGA for both the horizontal and vertical seismic coefficients used in the slope stability modeling. RCOL2_02.05.05-1 RCOL2_02.05.04-26 stability factors of safety, provided that the compacted fill materials exhibit the specified effective cohesion value of at least 200 psf, and an effective friction angle of at least 32 degrees, in accordance with the engineered fill specification.

Factors of safety are summarized in Table 2.5.5-203. The estimated factors of safety for permanent slopes satisfy the minimum required value.

Pseudo-static factors of safety were estimated using horizontal and vertical acceleration coefficients equal to 0.1g. The resulting factors of safety range between 1.45~~21~~ and ~~5.58~~6.02 (Table 2.5.5-203) and are considerably greater than the required minimum value of 1.1. These results demonstrate that the seismic performance of analyzed cross sections is acceptable and that no seismically induced permanent slope displacement is expected at CPNPP Units 3 and 4 site. RCOL2_02.05.04-26 RCOL2_02.05.05-1

In areas where undocumented fill or weak shale materials daylight within the slopes such as Cross Sections F-F', G-G', and H-H', an engineered buttress consisting of compacted fill founded into limestone Layer C or other competent material is required to maintain an acceptable stability performance.

A liquefaction potential evaluation, as discussed in Subsection 2.5.4.8, indicates that the native rock material supporting all seismic category I and II structures and the engineered compacted fill surrounding the structures are not susceptible to soil liquefaction and there is no impact on any safety related structures.

The post-construction cut slopes around the west and south periphery of the CPNPP Units 3 and 4 site presented in Table 2.5.5-201 and shown on Figure 2.5.5-204, are not considered to pose any slope stability issues or hazards to seismic category I and II structures. The closest approach between the toe of the cut slopes and seismic category I or II structures is approximately 150 ft, with a minimum ratio of at least three times the height of slope, providing a substantial safety setback from the cut slopes. Additionally, the inclination of cut slopes is generally 2(H):1(V) or flatter. Considering the strength properties of the materials comprising the cut slopes (residual soil over Glen Rose Formation rock) and the maximum inclination of 2(H):1(V), all these cut slopes are considered to be inherently stable.

All safety-related plant structures are supported by foundations bearing into the competent Glen Rose Formation Layer C limestone below the plant grade at elevation of about 782 ft, and do not use any of the adjacent slopes or embankments for support. As a result, embankments or fill slopes around the perimeter of the plant do not affect the stability or performance of the safety-related structures.

2.5.5.3 Logs of Borings

The slope stability analyses incorporated relevant exploratory boring information, and derivative laboratory test data from these borehole samples, as described in

Table 2.5.4-228
Summary of Ultimate Bearing Capacities

Structure	Category	Foundation Size (ft)		Foundation Bottom Elev. (ft)	Ultimate Bearing Capacity (ksf)		
		E-W	N-S		General Shear	Local Shear	Compression
R/B Complex	I	~~213~~347	~~309~~334.58	~~783~~779.75	~~354~~367	~~348~~364	146
T/B	II	~~186~~265.5	~~315~~342.67	~~795~~794.83	~~342~~356	~~339~~354	146
T/G	II	62.33	233.42	786.83	318	315	146
~~A/B~~	~~II~~	~~133~~	~~239~~	~~785~~	~~338~~	~~335~~	~~146~~
~~EPS/B~~	~~I~~	~~115~~	~~69~~	~~785~~	~~343~~	~~340~~	~~146~~
~~WPS/B~~	~~I~~	~~115~~	~~69~~	~~785~~	~~343~~	~~340~~	~~146~~
PSFSV	I	~~85~~98	~~78~~95	782	~~365~~367	~~362~~364	146
UHSRS	I	~~131~~267	~~131~~160	~~787~~786	~~369~~343	~~365~~340	146

RCOL2_02.
05.04-17

Table 2.5.4-229
Summary of Settlement Estimates Based on “BE” Profile

Structure	Category	Foundation Size (ft)		Foundation Bottom Elev. (ft)	Foundation Static Load (ksf)	Settlement Estimate for Center (in)	
		E-W	N-S			Non-Layered Method	Layered Method
R/B Complex	I	~~213~~347	~~309~~334.58	~~783~~779.75	~~11.3~~13.1	~~0.12~~0.17	~~0.20~~0.29
~~T/B~~	~~II~~	~~186~~	~~315~~	~~795~~	~~5.9~~	~~0.07~~	~~0.11~~
~~A/B~~	~~II~~	~~133~~	~~239~~	~~785~~	~~6.8~~	~~0.09~~	~~0.14~~
~~EPS/B~~	~~I~~	~~115~~	~~69~~	~~785~~	~~4.3~~	~~0.07~~	~~0.10~~
~~WPS/B~~	~~I~~	~~115~~	~~69~~	~~785~~	~~4.3~~	~~0.08~~	~~0.12~~
PSFSV	I	~~85~~98	~~78~~95	782	~~5.4~~4.0	~~0.06~~0.10-0.12	~~0.09~~0.13-0.17
UHSRS	I	~~131~~267	~~131~~160	~~787~~786	~~3.6~~4.9	~~0.05~~0.08	~~0.06~~0.10

RCOL2_02.05.04-17

Table 2.5.4-230
Summary of Settlement Estimates Based on “LB” Profile

Structure	Category	Foundation Size (ft)		Foundation Bottom Elev. (ft)	Foundation Static Load (ksf)	Settlement Estimate for Center (in)	
		E-W	N-S			Non-Layered Method	Layered Method
R/B Complex	I	~~213~~347	~~309~~334.58	~~783~~779.75	~~11.3~~13.1	~~0.30~~0.45	~~0.37~~0.52
~~T/B~~	~~II~~	~~186~~	~~315~~	~~795~~	~~5.9~~	~~0.19~~	~~0.20~~
~~A/B~~	~~II~~	~~133~~	~~239~~	~~785~~	~~6.8~~	~~0.23~~	~~0.26~~
~~EPS/B~~	~~I~~	~~115~~	~~69~~	~~785~~	~~4.3~~	~~0.18~~	~~0.18~~
~~WPS/B~~	~~I~~	~~115~~	~~69~~	~~785~~	~~4.3~~	~~0.20~~	~~0.21~~
PSFSV	I	~~85~~98	~~78~~95	782	~~5.4~~4.0	~~0.17~~0.26-0.30	~~0.16~~0.22-0.28
UHSRS	I	~~131~~267	~~131~~160	~~787~~786	~~3.6~~4.9	~~0.14~~0.22	~~0.12~~0.18

RCOL2_02.
05.04-17

Table 2.5.4-231
Summary of Rebound Estimates Based on "BE" Profile

Structure	Category	Foundation Size (ft)		Excavation Depth (ft)	Rebound Estimates for Center (in)	
		E-W	N-S		Non-Layered Method	Layered Method
R/B Complex	I	~~213~~347	~~309~~334.58	~~40-50~~40-50	~~0.07~~0.09	~~0.12~~0.14
~~T/B~~	~~II~~	~~186~~	~~315~~	~~40-50~~	~~0.06~~	~~0.10~~
~~A/B~~	~~II~~	~~133~~	~~239~~	~~40-50~~	~~0.07~~	~~0.10~~
~~EPS/B~~	~~I~~	~~115~~	~~69~~	~~40-50~~	~~0.06~~	~~0.08~~
~~WPS/B~~	~~I~~	~~115~~	~~69~~	~~40-50~~	~~0.06~~	~~0.10~~
PSFSV	I	~~85~~98	~~78~~95	~~40-50~~40-50	~~0.05~~0.06-0.07	~~0.08~~0.08-0.10
UHSRS	I	~~131~~267	~~131~~160	~~40-50~~40-50	~~0.05~~0.06	~~0.07~~0.09

RCOL2_02.
05.04-17

CP COL 2.5(1)

Table 2.5.5-201
Permanent Slopes Within CPNPP Units 3 and 4 Vicinity

Slope Location	Adjacent ~~Seismic~~ Category I Structure	Slope Type	Constructed Condition	Slope Orientation Relative to Yard Grade	Minimum Distance to Slope Crest/Toe	Slope Height[a]	Maximum Slope Inclination (H~~orizontal~~:V~~ertical~~)	RCOL2_02. 05.04-26 RCOL2_02. 05.04-26
Northwest of Unit 4	R/B Complex UHSRS	Fill	Engineered Fill, Residual Soil & Native Rock	Descending	360 ft to R/B Complex 100 ft to UHSRS	38 ft	2:1	
West of Unit 4	R/B Complex ~~WPS/B~~ PSFSV	Cut	Residual Soil & Native Rock	Ascending	~~330~~180 ft to R/B Complex ~~210 ft to WPS/B~~ 170 ft to PSFSV	20 ft	2:1	RCOL2_02. 05.04-26
Southwest of Unit 4	R/B Complex PSFSV	Cut	Residual Soil & Native Rock	Ascending	~~420~~260 ft to R/B Complex 210 ft to PSFSV	30 ft	2.5:1	RCOL2_02. 05.04-26
South of Unit 4	R/B Complex PSFSV	Cut	Residual Soil & Native Rock	Ascending	540 ft to R/B Complex 420 ft to PSFSV	45 ft	2:1	RCOL2_02. 05.04-26
East-Northeast of Unit 3	R/B Complex EPS/B UHSRS	Cut	Engineered Fill, Residual Soil & Native Rock	Descending	250 ft to R/B Complex 130 ft to EPS/B 110 ft to UHSRS	15 ft	3:1	RCOL2_02. 05.04-26 RCOL2_02. 05.04-26
South of Unit 3	R/B Complex PSFSV	Cut	Residual Soil & Native Rock	Ascending	580 ft to R/B 500 ft to PSFSV	20 ft	2:1	

a) Slope heights are determined with respect to yard grade elevation of 822 ft for ascending slopes and with respect to Squaw Creek Lake elevation level of 775 ft for descending slopes. RCOL2_02 05.04-26

CP COL 2.5(1)

Table 2.5.5-203
Summary of Stability Analyses

Cases	Cross Section	Static Slope Stability Factor of Safety	Pseudo-static Slope Stability Factor of Safety	
Permanent	D-D'	8.23~~1~~	~~5.58~~6.02	RCOL2_02.05.04-26 RCOL2_02.05.05-1
Permanent	E-E'	6.35~~29~~	5.26	
Permanent	F-F'	1.97	1.49	
Permanent	G-G'	1.89	1.47	
Permanent	H-H'	~~1.99~~2.39	1.45~~21~~	RCOL2_02.05.04-26 RCOL2_02.05.05-1