2.0 SITE CHARACTERISTICS

2.5.4 Stability of Subsurface Materials and Foundations

2.5.4.1 Introduction

The Tennessee Valley Authority (TVA) Early Site Permit (ESP) Site Safety Analysis Report (SSAR), Revision 1 (TVA, 2017 - Agencywide Documents Access and Management System (ADAMS) Accession No. ML18003A374), Section 2.5.4, "Stabiltiy of Subsurface Materials and Foundations," presents an evaluation of the stability of subsurface materials and foundations that relate to the Clinch River Nuclear (CRN) Site. The properties and stability of the soil and rock underlying the site are important to the safe design and siting of the plant. The information provided by the applicant in SSAR Section 2.5.4 addresses: (1) Geologic features in the site vicinity; (2) static and dynamic engineering properties of soil and rock strata underlying the site; (3) the relationship of the foundations for safety-related facilities and the engineering properties of underlying materials; (4) results of geophysical surveys, including in-hole and cross-hole explorations; (5) safety-related excavation and backfill plans and engineered earthwork analysis and criteria; (6) groundwater conditions and piezometric pressure in all critical strata as they affect the loading and stability of foundation materials; (7) responses of site soils or rocks to dynamic loading; (8) liquefaction potential and consequences of liquefaction of all subsurface soils, including the settlement of foundations; (9) earthquake design bases; (10) evaluation of static and dynamic stability of safety-related structure foundations including bearing capacity, heave, settlement, and lateral earth; (11) criteria, references, and design methods used in static and seismic analyses of foundation materials; (12) techniques and specifications to improve subsurface conditions, which are to be used at the CRN Site to provide suitable foundation conditions; and any additional information provided by the applicant in accordance with 10 CFR Part 52, "Licenses, Certifications; and Approvals for Nuclear Power Plants."

As discussed below, the staff concludes that the applicant has provided sufficient information to characterize the stability of subsurface materials and foundations for the CRN early site permit (ESP) application. However, additional site investigation activities need to be performed by a future combined license (COL) or construction permit (CP) applicant after selection of a specific reactor technology and location. The staff has identified 16 COL action items to document these additional activities. The COL action items would be resolved in a future COL or CP application.

2.5.4.2 Summary of Application

In SSAR Section 2.5.4, the applicant presented information on the stability of subsurface materials and foundations at the CRN Site based on the results of site geological, geophysical, and geotechnical investigations. The applicant has not selected a reactor technology to be constructed at the CRN Site. The applicant identified a set of bounding parameters using available information from four light-water-cooled, small modular reactor (SMR) designs to develop the plant parameter envelope (PPE). SSAR Table 2.0-1 provides a summary of the site characteristics at the CRN Site, and Table 2.0-2 provides site related design parameters from the PPE.

The applicant originally planned the CRN Site to support a CP application and identified two locations for the units considered at that time. Those locations are identified in the CRN Site ESP application as Location A and Location B. The applicant performed the subsurface investigations over a substantial portion of the CRN Site but predominantly within the footprint of the power block area. In this safety evaluation report (SER), Figure 2.5.4-1, "Geotechnical Cross-Section of the Stratigraphy of the Power Block Area," shows a cross-section through the power block area that illustrates the approximate ground surface and site stratigraphy including locations A and B.

The applicant stated that additional site-specific exploration and testing required to support the COL application will be performed when a reactor technology is selected.

2.5.4.2.1 Geologic Features

SSAR Section 2.5.4.1 refers to SSAR Subsection 2.5.1 for a detailed description of geologic features at the CRN Site. The applicant described the existing site elevations in the power block area as ranging from approximately 260.6 to 237.7 meters (m) (855 to 780 feet (ft)) with an average elevation of 246.9 m (810 ft) North American Vertical Datum (NAVD88). The applicant stated that a finished plant grade elevation will be at 250.2 m (821 ft) and foundation embedment is not expected to exceed elevation 208.2 m (683 ft) NAVD88. All references to elevations specified in this report are to NAVD88, with the exception of elevations pertaining to the Clinch River Breeder Reactor Project (CRBRP) which are with respect to the National Geodetic Vertical Datum of 1929 (NGVD29).

2.5.4.2.1.1 Stratigraphy

SSAR 2.5.4.1.1 refers to SSAR Sections 2.5.1.2.3.2 and 2.5.1.1.3.1 for a complete description of the stratigraphy of the site. The applicant stated that the stratigraphic units at the site strike northeast and dip relatively steeply to the southeast. Figure 2.5.4-1 (SSAR Figure 2.5.4-2) shows a cross-section of the stratigraphic units at the site underneath the power block area. The applicant identified the stratigraphic units underlying the power block area as predominantly the Newala Formation, belonging to the Knox Group, the Blackford Formation, the Lincolnshire Formation (Eidson and Fleanor Members), and Rockdell and Benbolt Formations belonging to the Chickamauga Group. The applicant stated that the contact between the Knox and the Chickamauga groups is an unconformity.

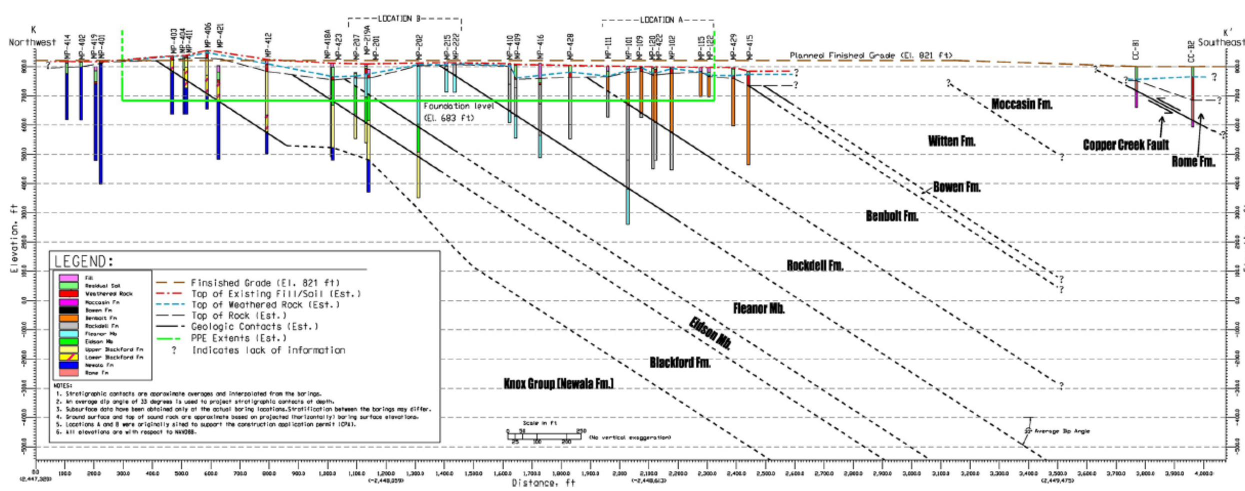


Figure 2.5.4-1: Geotechnical Cross-Section of the Stratigraphy of the Power Block Area (Reproduced from SSAR Figure 2.5.4-2)

The applicant used acoustic televiewer (ATV) logging and outcrop mapping to estimate the average strike and dip of the bedding planes for the units. The applicant stated that the average strike and dip of the bedding planes is N63°E and 33°SE and that it does not change considerably between stratigraphic units. The applicant used a dip angle of 33 degrees to project the contacts between the stratigraphic units at depth in the power block area, and to estimate the vertical thickness of each stratigraphic unit. The applicant noted that due to the dipping beds found at the site that various units may be exposed at the foundation elevation (El. 208.2 m (683 ft)) when the future excavation of geolocic material is completed.

2.5.4.2.1.2 Previous Loading History

In SSAR Section 2.5.4.1.2, the applicant indicated that the CRN Site area has undergone extensive periods of excavation, backfilling, grading and redressing associated with the Clinch River Breeder Reactor Project (CRBRP). The applicant noted that towards the center of the CRN Site, two (2) hills were removed by blasting techniques and an excavation of approximately 30.5 m (100 ft) below the ground surface was done for the reactor buildings of the now abandoned CRBRP. The applicant stated that up to about 6.1 m (20 ft) of fill was placed in the southern portion of the power block area and up to about 21.3 m (70 ft) of material was removed from the central and northern portions.

2.5.4.2.1.3 Discontinuities, Shear-Fracture Zones, Weathered/Fracture Zones

SSAR Section 2.5.4.1.3.1 summarizes discontinuities encountered at the CRN Site and refers to SSAR Subsection 2.5.1.2.4 for more details on bedding planes and joints. The applicant identified two primary joint sets; Joint Set 1 and Joint Set 2. Joint Set 1 has an average strike and dip of N60°E and 59°NW, and Joint Set 2 has an average strike and dip of N60°E and 38°SE. The applicant stated that these joint sets strike parallel to the strike of the bedding planes, which have an average strike and dip of N63°E and 33°SE. Additionally, the applicant identified three near-vertical secondary joint sets, one striking parallel to the strike of the bedding and the remaining two striking parallel to the bedding. The applicant stated that the highest frequency of joints occur within the upper 30.5 m (100 ft) of bedrock. In addition, the applicant indicated that the two primary joints sets are prevalent in all stratigraphic units, where the secondary joints are found predominantly in the Newala Formation. The applicant described the condition of the joints as undulating to planar, rough to smooth to slickensided, very tight to open with tightly healed to slightly altered joint walls and partially or wholly filled with calcite.

SSAR Section 2.5.4.1.3.2 summarizes shear-fracture zones encountered at the CRN Site and refers to SSAR Sections 2.5.1.2.4 and 2.5.1.2.6.4 for more details. The applicant stated that shear-fracture zones were encountered in the Rockdell and Benbolt Formations; and the Eidson Member between elevations of about 228.6 and 137.2 m (750 and 450 ft). The applicant described them as typically zones of multiple, closely spaced, tightly healed, calcite filled fractures with apparent thicknesses ranging from 0.3 to 6.7 m (1 to 22 ft) with an average of 1.2 m (4 ft). The applicant indicated that these shear fractures zones are likely to be found at or below foundation level and that it incorporated them in the Geological Strength Index (GSI) rating for each stratigraphic unit for rock mass characterization. The applicant stated that during excavation for the power block area, detailed geologic mapping will provide further characterization of any shear-fracture zones encountered and that additional evaluation of shear-fracture zones will be performed for the COL application, once the reactor technology is selected.

SSAR Section 2.5.4.1.3.3 summarizes weathered and fracture zones encountered at the CRN Site and refers to SSAR Section 2.5.1.2.6.3 for more details. The applicant stated that fracture zones typically occur along bedding planes or fractures and likely represent early dissolution of the limestone. In addition, the applicant characterized them as zones of poor to fair quality rock with slightly to highly weathered fractures or bedding planes. The applicant stated that these zones are mostly located within the first 15.2 m (50 ft) of the current ground surface (between elevation 243.8 and 228.6 m (800 and 750 ft)) with thicknesses ranging from 0.3 to 3.7 m (1 to 12 ft) with an average of about 0.9 m (3 ft). The applicant stated that further evaluation of weathered and fracture zones will be performed for the COL application, once the reactor technology is selected.

2.5.4.2.1.4 Karst Features

SSAR Section 2.5.4.1.4 summarizes karst features encountered at the CRN Site and refers to SSAR Sections 2.5.1.2.5 for more details. The applicant stated that cavities are present in all of the stratigraphic units at the site but are more predominant in the Rockdell Formation and Eidson Member. The applicant stated that these cavities range from 0.3 to about 5.2 m (1 to about 17 ft) in height, include open and clay-filled voids and are predominantly found within the first 30.5 m (100 ft) of the current ground surface. The applicant noted that approximately four voids were encountered within 1.5 to 6.1 m (5 to 20 ft) below the deepest foundation embedment elevation of 208.2 m (683 ft) with range of heights from 0.2 to 1.3 m (0.7 to 4.3 ft). The applicant referred to SSAR Section 2.5.1.2.6.10 for a discussion of a mitigation plan to address possible cavities encountered at and below the foundation levels for safety-related structures during excavation. The applicant indicated that details of this plan will be developed further to support a future COL application.

2.5.4.2.1.5 Unrelieved Stresses in Bedrock

SSAR Section 2.5.4.1.5 summarizes unrelieved stresses in bedrock at the CRN Site and refers to SSAR Subsection 2.5.1.2.6 for more details. The applicant stated that high residual stresses are not expected or considered to be a hazard during construction or for bearing capacity of the foundation rock mass. The applicant noted that blasting techniques are expected at the site in order to remove overburden thus creating a disturbed zone of rock adjacent to the foundation. The applicant stated that this disturbance is accounted for in the rock mass strength properties.

2.5.4.2.2 Properties of Subsurface Materials

In SSAR 2.5.4.2, the applicant described the static and dynamic engineering properties of the CRN Site subsurface materials, including field investigations, laboratory tests, and engineering properties determined from subsurface exploration activities.

2.5.4.2.2.1 Description of Subsurface Materials

SSAR Subsection 2.5.4.2.1.1 briefly describes the existing fill and residual soils at the CRN Site. The applicant stated that both the existing fill and residual soils are classified as high plasticity (CH) clays according to the Unified Soil Classification System (USCS) with median Standard Penetration Test (SPT) N_{60}-values of 14 and 19 blows per foot (bpf), respectively. The thickest deposits for both soils that the applicant encountered is 15.5 m (51 ft).

SSAR Subsection 2.5.4.2.1.2 briefly describes the new backfill to be placed at the CRN Site and refers to SSAR Section 2.5.4.5 for more details. The applicant stated that both lean concrete and granular backfill will surround the safety-related structures at the CRN Site. The applicant indicated that lean concrete will extend from the foundation level to the top of the rock. Granular backfill will be used from the top of the rock to the finished grade.

SSAR Subsection 2.5.4.2.1.3 describes the weathered rock found at the CRN Site. The applicant initially defined weathered rock as material having a SPT blow count of 50 bpf, which results in less than 0.2 m (6 inches (in.)) of penetration. The applicant indicated that weathered rock is encountered in most of the borings drilled at the site. The applicant used different methods to define the thickness of the weathered rock and subsequent depth to bedrock throughout the site, including: Rock Quality Designation (RQD) values, shear wave velocity (V_s) values, drill rates, and rock core photographs. The applicant stated that the maximum thickness of the weathered rock at the site is approximately 11.9 m (39 ft) and that the weathered rock will be excavated from the power block area prior to construction of foundations.

In SSAR Subsections 2.5.4.2.1.4 through 2.5.4.2.1.10, the applicant described the stratigraphic units encountered at the site. Safety Evalution Report (SER) Table 2.5.4-1, summarizes some of the properties of the rock stratigraphic units at the CRN Site.

Rock Strata	Description	Vertical Thickness m (ft)	Average RQD (%)
Bowen Formation	Reddish brown to olive brown, laminated to very thinly bedded calcareous siltstone.	9.1 (30)	26
Benbolt Formation	Gray limestone (micrite/wackestone), strong, very thinly to thinly bedded, locally moderately bedded, and nodular limestone interbedded with little to some laminated to thinly bedded calcareous siltstone.	100.6 (330)	88
Rockdell Formation	Gray and brownish-gray, strong, laminated to moderately bedded limestone (micrite/wackestone/grainstone), interbedded with few to little, laminated to very thinly bedded calcareous siltstone.	87.5 (287)	88
Fleanor Member (Lincolnshire Formation)	Red, medium strong, laminated to medium bedded, calcareous siltstone with few to little gray micritic limestone layers.	78.3 (257)	89
Eidson Member (Lincolnshire Formation)	Gray, medium strong and strong, laminated to thinly bedded, fresh, argillaceous micritic limestone.	31.1 (102)	80
Blackford Formation	The Lower Blackford is generally described as a gray, locally mottled, strong, laminated to thickly bedded, micritic limestone. The Upper Blackford is generally described as a gray, calcareous siltstone, laminated to moderately bedded, interbedded with little to some limestone with few to little chert beds, lenses and nodules.	77.4 (254)	81
Newala Formation	Fresh, fine to medium grained, gray, locally mottled red, strong to very strong, moderately to thickly bedded crystalline dolomite, with few irregular chert nodules and chert beds.	-	93

"-"= unknown, none of the borings at the site penetrated the full thickness of the strata

Table 2.5.4-1 Summary of Properties of CRN Rock Stratigraphy

2.5.4.2.2.2 Field Investigations

SSAR Section 2.5.4.2.2 refers to SSAR Sections 2.5.4.3 and 2.5.4.4 for a description of the field investigation program and geophysical surveys performed for the CRBRP and the CRN Site. The applicant stated that the field investigation at the CRN Site was performed in accordance with guidance in RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants, Revision 2." (NRC 2003 – ADAMS Accession No. ML032790474)

2.5.4.2.2.3 Laboratory Testing

SSAR Section 2.5.4.2.3 provides a brief description of the applicant's laboratory testing. The applicant stated that the laboratory testing was performed in accordance to RG 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants, Revision 3" (NRC 2014 – ADAMS Accession No. ML14289A600) and under an approved quality assurance program. The applicant stated that the soil and rock samples were shipped under chain of custody protection from the storage area to the testing laboratory. The applicant indicated that the laboratory tests performed on the soil samples focused on obtaining the basic characteristics of the soils and the shear strength and compaction characteristics. The applicant stated that the tests performed on the rock core samples focused on obtaining the basic characteristics of the rock and the compressive strength, shear and elastic moduli, Poisson's ratio, slake durability and calcium carbonate content. The applicant stated that details and results of the laboratory testing are included in Appendices F through H of the Geotechnical and Exploration and Testing report (AMEC, 2014).

2.5.4.2.2.4 Engineering Properties

The applicant derived the engineering properties for the existing fill and residual soil, granular backfill, weathered rock, and the bedrock around the power block area from the CRN Site subsurface investigation and the laboratory testing program. SSAR Table 2.5.4-21, summarizes the selected values of the engineering properties for the materials beneath the power block area. The applicant developed the engineering properties to evaluate the stability of the foundation materials.

Soil Properties

The applicant recommended SPT N_{60} values based on corrected field measured N values. The applicant adjusted the field measured N values using energy correction factor, adjustment for field procedures, borehole diameter, and sampler correction factor and rod length correction. The applicant performed sieve analyses of 34 existing fill and residual soil samples. The applicant estimated the unconfined compressive strength based on SPT N_{60} values. The applicant classified the existing and residual soils as CH based on the USCS, American Society for Testing and Materials (ASTM) D2487. The applicant determined the undrained shear strength from unconsolidated undrained triaxial testing, and also estimated it from the unconfined compressive strength of the soil using the relationship that the undrained shear strength is approximately one-half the unconfined compressive strength. The applicant determined the drained shear strength, effective cohesion, and angle of internal friction from consolidated undrained (CU) triaxial tests. The applicant used the suspension P-S velocity method to record the shear (V_s) and compression (V_p) wave velocity measurements. The applicant calculated the Poisson's ration based on wave velocity measurements. The applicant derived the low strain shear modulus and low strain elastic modulus using the following relationships:

$$G_L = \left(\frac{\gamma}{g}\right) * V_s{}^2$$

$$E = 2\,(1 + \mu) * G$$

where;

G= shear modulus
E= elastic modulus
γ= total unit weight
μ= Poisson's ratio
g= acceleration due to gravity
V_s= shear wave velocity

The applicant used the following relationship with the undrained shear strength to derive the high strain or static modulus.

$$E_H = 600 * S_U$$

where;

E_H= high strain elastic modulus
S_u= undrained shear strength

The applicant determined the maximum dry density and optimum moisture content as part of the laboratory testing program. SSAR Table 2.5.4-21 contains all recommended values for soil properties.

Weathered Rock Properties

The applicant stated that the weathered rock will be excavated during construction. The applicant considered the weathered rock in site response analyses and selected engineering properties from in situ testing and material correlations.

Intact Rock Properties

In SSAR Section 2.5.4.2.4.3, the applicant described the properties of the intact rock underneath the power block area including: the Benbolt, Rockdell, Blackford and Newala Formations and the Fleanor and Eidson Members. The applicant determined the total unit weight and specific gravity from the laboratory test results. The applicant conducted moisture content testing on rock core samples from the Fleanor Member and conducted unconfined compressive strength tests as part of the laboratory testing program. In SSAR Table 2.5.4-16 presents a summary of the V_s and V_p measurements for each stratigraphic units. The applicant calculated the Poisson's ratio based on wave velocity measurements. The applicant derived the low strain shear modulus and low strain elastic modulus and high strain shear modulus using the same relationships it used for soils. The applicant stated that, for sound rock, the shear and elastic moduli typically remain constant at both small and large strains as indicated by the similar results for the low strain and high strain shear and elastic moduli of the stratigraphic units. The applicant stated that results from the pressuremeter testing indicate a strain hardening behavior. This suggests that the use of a low strain value is conservative. The applicant derived the coefficient of sliding from the tangent of the friction angle between foundation material and the bedrock. The applicant performed slake durability and calcium carbonate content test as part of the laboratory testing program. SSAR Table 2.5.4-21 contains all recommended values for the rock properties.

Rock Mass Properties

SSAR Section 2.5.4.2.4.4 describes the rock mass strength and deformation properties developed for the stratigraphic units encountered within the power block area. The applicant developed the rock mass properties using the GSI classification and the Hoek-Brown failure criterion, which assumes that the rock mass contains several sets of discontinuities that are closely spaced relative to the proposed structure, such that it behaves as a homogeneous and isotropic mass and that a predetermined failure plane does not exist. The applicant indicated that the size of the power block area excavation is expected to be much larger than the rock blocks that make up the rock mass at the site. The applicant stated that rock core and geophysical data regarding discontinuities and fracture zones were reviewed and that the data indicate that weathered and fractures zones are, for the most part, encountered in the uppermost 30.5 m (100 ft). The applicant stated, based on the observation from the grouting program and the excavation for the CRBRP, that the rock mass below this zone typically is tighter and contains less frequent and less persistent discontinuities.

SSAR Section 2.5.1.2.6 includes a detailed description of the GSI for each stratigraphic unit. The applicant stated that the rock mass at the CRN Site contains five distinct joint sets that define the blockiness of the rock mass, making the GSI classification system applicable to the site. In SSAR Table 2.5.1-15, summarizes the GSI results for each stratigraphic unit. The applicant used the GSI to estimate rock mass strength and deformation properties. The applicant developed the rock mass strength and deformation properties for the stratigraphic units within the disturbed zone adjacent to the foundation to account for stress relief and blast damage of the rock mass immediately adjacent to the foundation and the undisturbed zone. The applicant used the RocData computer program to determine the rock mass strength using the generalized Hoek-Brown criterion. The applicant used a disturbance factor of 0.7 for damage from controlled blasting. The applicant stated that when comparing rock mass compressive strength against intact compressive strength for the stratigraphic units with GSI greater than or equal to 80, the rock mass compressive strength between 10.3 and 45.5 megapascals (MPa) (1,500 and 6,600 pounds per square inch (psi)) are approximately one-third of the intact compressive strength of 31 to 137.9 MPa (4,500 to 20,000 psi).

The applicant developed the deformation modulus of the rock mass using methods available in the RocData computer program and using empirical equations. In SSAR Table 2.5.4-25, summarizes the rock mass deformation moduli estimated using empirical equations and the modulus obtained from in situ pressuremeter tests, and developed from the low strain V_s data for comparison purposes. The applicant indicated that rock mass deformation moduli for low strain are frequently overestimated using V_s data and frequently underestimated using the in situ pressumeter test method. The applicant indicated that the deformation moduli, derived from the V_s, range from approximately 34,473 to 78,600 MPa (5,000 to 11,400 kip per square inch (ksi)). Deformation moduli derived from in situ pressuremeter testing range from about 6,205 to 16,547 MPa (900 to 2,400 ksi). The applicant stated that the estimated moduli from the empirical equations generally occur between these ranges.

2.5.4.2.3 Foundation Interfaces

In SSAR Section 2.5.4.3, the applicant described the foundation interface conditions at the CRN Site and described geotechnical exploration and testing activities. The applicant summarized the subsurface investigation programs performed for the CRBRP and for the CRN Site. The applicant stated that the field investigations for determining the engineering properties of soil materials follow the guidance of RG 1.132.

The CRN Site subsurface investigation included 82 exploratory borings, three test pits, 44 observation wells, two surface geophysical tests – reflection and refraction, downhole geophysical tests in 28 borings, field permeability and pumping tests, and groundwater level monitoring in the observation wells.

Figure 2.5.4-1 (SSAR Figure 2.5.4-2) presents a cross-section illustrating the position of subsurface stratigraphy with the assumed foundation elevation for safety-related structures within the PPE.

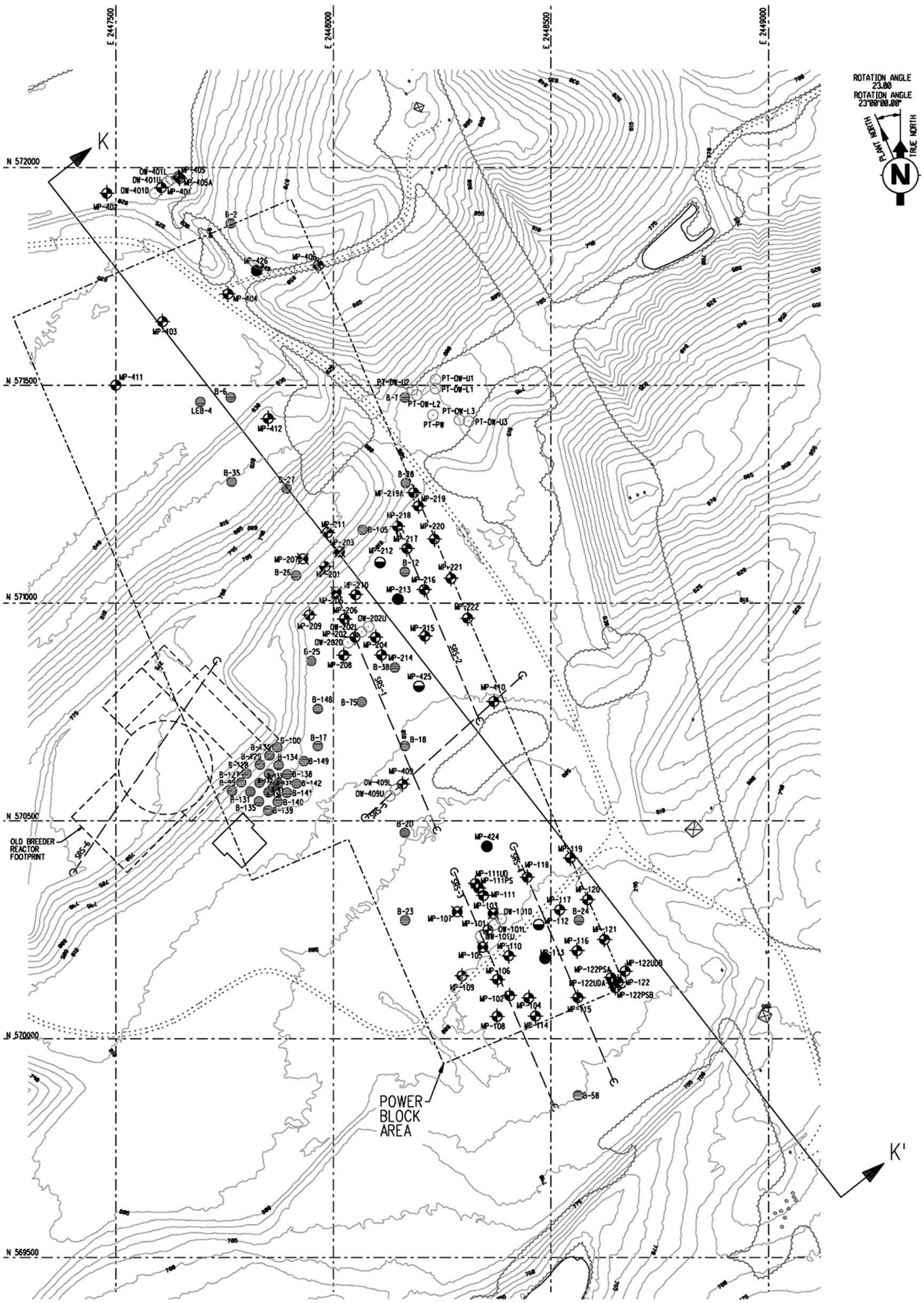
2.5.4.2.3.1 Borings and Soil Samples/Rock Cores

The applicant drilled 82 borings at the CRN Site from depths of about 6.1 to 164.6 m (20 to 540 ft). The deep boreholes were at least 61.0 m (200 ft) deeper than the deepest foundation embedment depth in the PPE. Seven of the borings were drilled at inclinations of between 25 and 29 degrees from the vertical. All borings were advanced until SPT refusal. The applicant followed the guidance in RG 1.132 for the sampling interval, and ASTM standards when conducting SPT and collecting samples. Figure 2.5.4-2 (SSAR Figure 2.5.4-1) shows the boreholes at the CRN Site within or near thePower Block area.

2.5.4.2.3.2 Groundwater Observation Wells and Associated Tests

In SSAR Sections 2.5.4.3.2.2 and 2.5.4.3.2.3, the applicant described the groundwater wells installed at the CRN Site, along with associated tests. The applicant stated that 44 groundwater wells were installed at the CRN Site with 34 of the wells used as groundwater observation wells for monitoring of groundwater levels and for water quality sampling in select wells. The additional wells were for aquifer performance testing, which consist of one pumping test well, and six adjacent pumping-test-specific observation wells.

The applicant placed the observation wells in the weathered rock and/or bedrock between elevations of about 242.6 and 150.0 m (796 and 492 ft), and each well was developed by pumping and surging. Field permeability slug tests were performed in most of the observation wells to obtain estimates of transmissivity, storage coefficient, and hydraulic conductivity in accordance with ASTM D4044. Both rising and falling head tests were performed when possible.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E 2448000
E 2448500
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N 572000
N 571500
N 571000
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N 569500
ROTATION ANGLE 23.00
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PLANT NORTH
TRUE NORTH
N
K
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OLD BREEDER REACTOR FOOTPRINT
POWER BLOCK AREA
SRS-1
SRS-2
SRS-5
SRS-6
MP-402
MP-403
MP-404
MP-405
MP-405A
MP-406
MP-401
OW-401L
OW-401U
OW-401D
MP-411
MP-412
MP-426
LEB-4
B-2
B-6
B-7
B-35
B-27
B-28
PT-OW-U1
PT-OW-L1
PT-OW-U2
PT-OW-L2
PT-OW-L3
PT-OW-U3
PT-PW
MP-219A
MP-219
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MP-211
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MP-207
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MP-221
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MP-213
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MP-208
MP-425
MP-410
MP-409
OW-202U
OW-202L
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B-105
B-12
B-26
B-25
B-38
B-148
B-75
B-17
B-18
B-100
B-134
B-149
B-138
B-142
B-141
B-140
B-139
B-135
B-131
B-20
B-23
B-24
B-58
MP-424
MP-119
MP-118
MP-111UD
MP-111PS
MP-111
MP-103
MP-107
MP-101
OW-101D
OW-101L
OW-101U
MP-117
MP-112
MP-120
MP-121
MP-116
MP-110
MP-105
MP-113
MP-106
MP-109
MP-122PSA
MP-122UDB
MP-122UDA
MP-122
MP-122PSB
MP-102
MP-104
MP-114
MP-108
MP-115

Figure 2.5.4-2: Boring Location Plan at the CRN Site (Reproduced from SSAR Figure 2.5.4-1)

The applicant stated that water level measurements were initially taken in the completed observations wells on a weekly basis for three months. Measurements were then collected on a monthly basis for the remainder of the 12-month period, followed by collection on a quarterly basis for the second year of monitoring. Pressure transducers were installed in 13 of the observation wells for continuous groundwater level monitoring. Groundwater samples were obtained from selected observation wells for geochemical characterization including pH, specific conductance, turbidity, dissolved oxygen, temperature, redox potential, and for major anions and cations.

2.5.4.2.3.3 Test Pits

The applicant excavated three test pits at the CRN Site with two of the pits located in the footprint of the power block. The test pits were used to visually describe and classify soil in the field and to obtain bulk samples of representative soil types. The test pits were then backfilled with the excavated soil after test completion.

2.5.4.2.3.4 Rock Tests

The applicant performed rock pressuremeter and direct shear tests. The pressuremeter tests were performed in two borings for rocks within the Benbolt and Rockdell formations, and the Fleanor and Eidson Members. Direct shear strength tests were performed on nine rock core samples in accordance with ASTM D5607, which included intact shear strength tests on five rock core samples and sliding friction tests on four rock core samples.

The applicant stated that Goodman Jack in situ tests (a borehole test to determining the in situ modulus of deformation of rock) were conducted for the CRBRP. The range of elastic moduli values derived from these tests for the Fleanor and Eidson Members is larger than those determined from the pressuremeter test results at the CRN Site.

2.5.4.2.4 Geophysical Surveys

SSAR Section 2.5.4.4 describes the geophysical surveys that the applicant conducted in its site investigation at the CRN Site. These surveys consist of surface and downhole geophysical testing that includes seismic refraction and reflection surveys. The surveys also consist of a suite of downhole tests including Suspension P-S velocity logging to obtain Vp and Vs measurements; other borehole loggings; acoustic televiewer (ATV), conductivity and natural-gamma data for soil, rock and fluid; and fluid temperature data, along with borehole deviation measurement. The applicant also provided a summary of geophysical surveys for the CRBRP site and compared those results with the data obtained from the geophysical surveys at the CRN Site.

2.5.4.2.4.1 Surface Geophysical Testing

The surface geophysical testing at the CRN Site included seismic refraction and reflection surveys. The seismic refraction survey was conducted to map the depth to bedrock beneath six seismic refraction profiles using the P-wave seismic refraction technique. The survey showed that the interpreted depth to bedrock is between approximately 2.7 and 12.8 m (9 and 42 ft) at the central portion of the power block area. In the southern portion of the power block area, the interpreted depth to bedrock is between approximately 4.9 and 13.1 m (16 and 43 ft), which is reasonably consistent with that observed in the boring logs. The applicant considered that any differences in the depth to bedrock between the interpreted seismic-bedrock interface and the observations from the boring logs reflect the degree of weathering of the bedrock and/or the presence of saturated soil. The tomographic model for the former excavation for the CRBRP shows that the interpreted depth to bedrock is about 16.5 m (54 ft), with shallower depths of about 0.9 and 9.1 m (3 and 30 ft) beneath the westernmost and easternmost portions of the refraction profile lines, respectively.

The applicant conducted a seismic reflection survey to interpret the contact (disconformity) between the stratigraphic units of the Chickamauga Group and underlying Knox Group; the general inclination of the bedding planes in the stratigraphic units between the borings; and the presence of any anomalies, such as faults or cavities. Two survey lines were placed during the reflection survey: one line was located in the power block area, and the other was located west of the power block area. The applicant stated that the seismic reflection survey was conducted using the P-wave seismic reflection technique based on the procedure outlined in ASTM D7128. The applicant stated that the seismic reflection survey data showed generally continuous, moderately steeply dipping rock beds. The applicant stated that the seismic reflection survey data showed generally continuous, moderately steeply dipping rock beds. The applicant identified three anomalous zones on the section within the power block area and two anomalous zones on the section west of the power block area. The applicant interpreted these anomalous areas as either being artifacts associated with out-of-plane reflectors or special aliasing, or as representing the effects of tuning or the interference from events outside of the plane of the seismic profile. The survey data identified no fault-like features.

2.5.4.2.4.2 Downhole Geophysical Testing

The applicant performed downhole geophysical testing in 27 uncased and 3 cased borings to measure Vp and Vs, deviation data, conductivity and natural-gamma data, caliper and natural-gamma data, fluid temperature, and fluid conductivity and natural-gamma data. It is noted that only downhole P-S logging and deviation testing were performed in the overburden in a select number of borings as the upper portions of those borings collapsed.

Suspension P-S Velocity Logging

The suspension P-S velocity logging was used to obtain in situ measurements of vertically propagating horizontally polarized shear and compressional wave velocities at 0.5 m (1.64 ft) intervals. The applicant processed the data and grouped the velocity measurements according to the stratigraphic unit based on their recorded mid-point depth in the boring and the stratigraphic contacts identified for each unit. The applicant stated that the compilation of the profiles did not include velocity measurements from the inclined borings or from boring MP-420 that was considered too far from the power block area, and measurements within the weathered rock were also not included.

The suspension P-S velocity logging date showed that the Newala Formation exhibits the highest average Vs and Vp of 3,292 m/s (10,800 feet per second (fps)) and 6,066 m/s (19,900 fps), respectively. The Rockdell Formation and Eidson Member exhibit similar velocities with average Vs of 2,743 m/s (9000 fps) and Vp of about 5,182 m/s (17,000 fps). The Benbolt and

Blackford Formations also exhibit similar Vs and Vp with average Vs of 2,438 and 2,499 m/s (8,000 and 8,200 fps) and average Vp of 4,694 and 4,785 m/s (15,400 and 15,700 fps), respectively. The Fleanor Member exhibits the lowest average Vs and Vp of 2,195 and 4,420 m/s (7,200 fps and 14,500 fps), respectively.

The applicant also presented the minimum, maximum, and average Vs and Vp, obtained for the CRBRP, for the Fleanor and Eidson Members and for the Blackford Formation. The CRBRP data showed similar seismic velocity values as those for the CRN Site. The applicant stated that the velocity profiles, as presented in SSAR Figure 2.5.4-5 and SSAR Figure 2.5.4-6, show that Vs and Vp do not vary significantly with depth for each rock formation.

Acoustic Televiewer (ATV) Logging

The applicant used a HiRAT model High-Resolution Acoustic Televiewer probe (HIRAT) to obtain boring deviation/inclination data, and to collect images of the borings walls in accordance with ASTM D5753. The processed data in three dimensional plots present true dip and azimuth of the borehole. The dip and dip azimuths of the discontinuities collected from ATV logging are used to analyze the discontinuity orientations, prepare scatter and contour plots of the discontinuity poles, and determine discontinuity sets and their average orientations. The oriented images of borehole cores generated by the HIRAT provide visual information on subsurface material.

The applicant stated that the deviation data show that all of the borings were inclined 3 degrees or less from the vertical (with a mean dip of 1.3 degrees) and that the greatest error in depth due to this deviation was 2.4 cm in 1,768 cm (0.08 ft in 58 ft), or about 0.15 percent of depth.

Induction/Natural-Gamma; Caliper/Natural-Gamma; Fluid Temperature/Fluid Conductivity/Natural-Gamma Logging

The applicant conducted induction/natural-gamma (gamma) logging to identify the lithostratigraphic units at the CRN Site. The logging was performed in accordance with ASTM D5753, ASTM D6274, and ASTM D6726 using a DUIN model dual induction probe. Gamma logs provide a record of natural-gamma radiation emitted from the boring walls. Induction logs measure conductivity and high-resolution information on lithology when combined with gamma logs. The applicant stated that the processed data were measured along the boring axis for the inclined borings. However, mechanical caliper data in the inclined borings are not used because the weight of the probe prevented the opening of the caliper arms against the boring wall.

The applicant used caliper/natural-gamma (gamma) logging to measure the diameter of the boreholes and to identify anomalous structures in the walls of the borings such as cavities, fissures, etc. Caliper measurements were collected concurrently with natural-gamma emissions in accordance with ASTM D5753, ASTM D6167, and ASTM D6274, using a Model 3ACS 3-leg caliper probe. The applicant stated that the multiple parameter logs show that changes in conductivity correspond with changes in natural-gamma and that the natural-gamma data agree well with natural-gamma data collected with the caliper data. Gamma signatures are typically higher in mud-supported rocks such as mudstones and siltstones. The natural-gamma logs reveal that gamma signatures are highest in the Fleanor Member, followed by the Benbolt and Blackford Formations, and lowest in the Eidson Member and Rockdell and Newala Formations. Caliper logs show consistent gauge below the bedrock surface and also the presence of open and clay-filled fractures by an increase in boring diameter and corresponding increase in natural-gamma. Caliper and natural-gamma plots correspond well with changes in velocity.

The applicant also performed fluid temperature/fluid conductivity/natural-gamma logging to identify the lithostratigraphic units and the presence of salt or fresh groundwater (for observation well siting). Fluid temperature and conductivity measurements were collected concurrently with the natural-gamma emissions in accordance with ASTM D5753 and ASTM D6274 using a temperature, conductivity, and gamma probe. The applicant stated that fluid temperature and conductivity changes generally correspond with fractures identified on the acoustic televiewer logs.

2.5.4.2.5 Excavation and Backfill

SSAR Section 2.5.4.5 describes the extent of anticipated safety-related excavations, fills and slopes, excavation methods and stability, backfill sources, quality control, Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC), construction dewatering impacts, and retaining walls at the CRN Site.

2.5.4.2.5.1 Extent of Excavations, Fill and Slopes

Figure 2.5.4-1 of this report shows a cross-section through the power block area that illustrates the approximate ground surface and site stratigraphy. At the center of Locations A and B, the top of bedrock is encountered approximately 6.1 and 9.1 m (20 and 30 ft) below the existing ground surface. The applicant stated that the finished plant grade elevation for the power block area is set at EL 250.2 m (821 ft), and the bottom of the basemat of the most deeply embedded safety-related power block structures are expected not to exceed a depth of 42.1 m (138 ft) below finished grade.

The applicant stated that the construction of the basemat at these locations requires a substantial amount of excavation in both soil and rock. Excavation sidewalls are expected to be vertical or near-vertical due in part to the depth of excavation, requiring the use of surface mounted cranes. The lateral extents of the excavation are expected to be limited, on the order of 4.6 m (15 ft) beyond the exterior face of the perimeters walls to provide working room for construction and backfilling of the exterior walls. The floor of the excavation is expected to be irregular due to the different stratigraphic units that are encountered. Concrete will be used to establish a level grade, and for the base of the basemat of safety-related power block structures. The deepest location of the foundation is expected not to exceed 42.1 m (138 ft) below finished grade.

The applicant stated that concrete backfill and compacted granular backfill are needed to backfill the excavation. The concrete backfill will be used underneath the basemat and around the structure from the basemat to the top of rock, then compacted granular backfill will be used above the elevation of rock to finished grade. Compacted granular backfill will also be used for general site grading in the power block to raise the grade to finished plant grade elevation.

In accordance with the PPE, construction of the safety-related structures requires a temporary excavation on the order of approximately 36.6 m (120 ft) below the existing grade at Location A and 39.6 m (130 ft) below the existing grade at Location B. The excavation slopes are made in existing fill/residual soil, weathered rock, and bedrock. Design of the excavation and backfill will be done for the COL application.

2.5.4.2.5.2 Excavation Methods and Stability

In SSAR Section 2.5.4.5.2, the applicant discussed excavation methods and associated slope stability issues. The applicant stated that excavation in existing fill/residual soil can be done with conventional earthmoving equipment. Excavation must adhere to regulations from Occupational Safety and Health Administration, 29 CFR Part 1926, "Safety and Health Regulations for Construction." Depending on the excavation depth, the excavations in soil may include vertical cuts supported with tied-back sheet piles or soldier pile and lagging walls. The side slopes of the ramp for construction access made in soil can be excavated at slope angles of 2 (horizontal) to 1 (vertical).

The applicant stated that conventional excavating equipment can be used to excavate weathered rock that is about 2.7 and 3.0 m (9 and 10 ft) beneath the existing fill/residual soil at Locations A and B within the power block area. Groundwater is generally encountered within the weathered rock. Therefore, groundwater control will be required during excavation and for excavation support.

The applicant stated that the excavation of rock likely requires the use of controlled blasting techniques, as it did for the CRBRP. For the CRBRP excavation, to minimize rock excavation, and to provide crane access to the bottom of the excavation, 22.9-m (75 ft) high near-vertical rock slopes in the north, south and east portions of the excavation were required. The applicant provided a more detailed description of the controlled blasting techniques that consist of production and perimeter blasting. For the stability of the near-vertical rock slopes, the applicant stated that rock bolts were needed and the design will be based on information from geologic mapping of exposed rock surfaces. Furthermore, the slope movement and foundation performance was monitored with an extensive instrumentation program during and after the excavation.

The applicant stated that the blasting program for the CRN Site varies depending on where the safety-related structure(s) are located and in which stratigraphic unit they are embedded. The applicant also stated that for COL applications, additional subsurface data may be required to further characterize the underlying stratigraphic bedrock units for the final plant layout. Specific design of the excavation support system, including rock bolting, will be developed during detailed design.

2.5.4.2.5.3 Backfill Sources

In SSAR Section 2.5.4.5.3, the applicant described general requirements for backfill materials. For granular backfill materials, the applicant suggested the use of a processed graded aggregate that meets the gradation requirements of Type A aggregate of the Tennessee Department of Transportation Standard Specifications for Road and Bridge Construction Section 303, Table A2.6. The applicant also suggested setting up a crushing and blending plant onsite to produce the crushed aggregate to the required gradation specification. Otherwise, the graded aggregate needs to be imported from nearby quarries. The application defers a detailed field and laboratory test program to the COL application for evaluation of backfill sources and their engineering properties. However, the applicant specified that the test program should include gradation (grain size distribution), density, soundness, durability, strength, and the dynamic properties of the backfill. A test pad will be needed to establish placement and compaction methods. The applicant also specified that the granular backfill should be compacted to at least 95 percent of the maximum dry density, as determined by the modified Proctor test, and that the moisture content of the compacted fill should be within 3 percent of its optimum moisture content.

2.5.4.2.5.4 Quality Control and ITAAC

In SSAR Section 2.5.4.5.4, the applicant provided general requirements for backfill and subgrade quality control but it leaves details to the COL application, including identification of quality requirements and industry standards for safety-related backfill material and placement specifications, as well as ITAAC related to backfill.

The applicant stated that a quality assurance and quality control program for the backfill needs to be established to verify that the granular backfill is constructed to the design requirements. The application specifies that for limited earthwork, where fill is compacted with hand equipment, one density test is conducted for every 56.6 m^2 per meter (2,000 ft^2 per foot) of fill placed. Otherwise, field density tests are performed at minimum of one per 929.0 m^2 (10,000 ft^2) of fill placed, with at least one test per lift.

The applicant stated that the concrete fill mix design specification will be provided during the detailed design phase of the project. Field observations and tests need to be performed to verify that specified design parameters are reached.

The applicant stated that COL applicants need to perform visual inspection of the final bedrock excavation surface to confirm that material is in general conformance with the expected foundation materials based on boring logs. COL applicants also need to perform visual inspection of exposed bedrock foundation subgrade to confirm that cleaning and surface preparations are completed in accordance with the specification. Geologic mapping of the final exposed excavated bedrock surface is required before placement of concrete (dental) backfill and foundation concrete, and will be conducted under the guidelines of NUREG/CR-5738 (SSAR Reference 2.5.4-38).

2.5.4.2.6 Groundwater Conditions

SSAR Section 2.5.4.6 summarizes the groundwater conditions at the CRN Site. Additional details are described in SSAR Section 2.4.12.

2.5.4.2.6.1 Groundwater Measurements and Elevations

The applicant installed 44 observation wells in two- and three-well clusters with screened intervals of upper (between 4.6 to 32.0 m (15 to 105 ft)), lower (between 27.1 to 54.3 m (89 to 178 ft)) and deeper (between 53.6 to 90.5 m (176 to 297 ft)) zones. Three observation well clusters installed in the power block area exhibited groundwater level elevations ranging from approximately 243.8 to 224.9 m (800 to 738 ft) in the upper zone, 237.4 to 215.2 m (779 to 706 ft) in the lower zone, and 233.2 to 225.2 m (765 to 739 ft) in the deeper zone. The applicant generally observed groundwater at depths ranging from near-surface to approximately 7.6 m (25 ft) below ground in the observation wells.

The applicant stated that the weathered rock generally acts as a water table aquifer and that most of the groundwater flow occurs within this zone. Groundwater flow also occurs through discontinuities and openings in the underlying bedrock, predominantly in the upper 30.5 to 45.7 m (100 to 150 ft) of the bedrock. The groundwater movement at the site is generally to the southeast and southwest towards the Clinch River arm of the Watts Bar Reservoir. Horizontal hydraulic gradients range from 0.03 to 0.11 m/m (ft/ft) and average vertical hydraulic gradients range from -0.71 m/m (ft/ft) (upward) to 1.15 m/m (ft/ft) (downward) for the observation well clusters. The applicant summarized the hydraulic conductivity values for the bedrock stratigraphic units based on the results of the slug tests. The applicant referred to details provided in SSAR Subsection 2.4.12.

2.5.4.2.6.2 Construction Dewatering

The applicant stated that, during construction, the groundwater levels at the site are likely to result in the need for temporary dewatering of the foundation excavations extending below the water table. The applicant suggested the use of gravity-type dewatering systems and the extraction of water using sump pumps in the lowest working levels of the excavation and then transfer to an impoundment facility. The applicant pointed out that dewatering should consider minimization of drawdown effects on the surrounding environment, and that appropriate methods should be used for open bedding planes and fractures to reduce groundwater inflow to the excavation and to reduce the extent of dewatering. Horizontal relief wells may be needed in the rock excavation walls to prevent hydrostatic pressure buildup behind the walls. The applicant also stated that the response to groundwater extraction needs to be assessed using a network of observation wells installed at the site, plus stream gauges when needed.

2.5.4.2.6.3 Groundwater Chemical Properties

SSAR Section 2.5.4.6.3 summarizes the groundwater chemical properties based on the geochemical test results for the CRN Site. The applicant stated that the pH of the groundwater ranges from 6.97 to 9.58 with an average pH of 7.53. The sulfate concentration of the groundwater ranges from 6.9 milligrams per liter (mg/L) to 150 mg/L with an average sulfate concentration of 42 mg/L. The chloride concentration ranges from 1.3 mg/L to 24 mg/L with an average chloride concentration of 4.5 mg/L. The applicant stated that with a sulfate concentration of 42 mg/L, the water-soluble sulfate concentration in contact with concrete is low and injurious sulfate attack is not a concern.

The applicant stated that for concrete fill and foundations, an Exposure Category C1 is assigned because the foundations will be exposed to moisture but will not be in contact with external sources of chlorides. Therefore, the applicant stated that the protection requirement specified in American Concrete Institute (ACI) standard 318-14 should be followed.

2.5.4.2.7 Response of Soil and Rock to Dynamic Loading

SSAR Section 2.5.4.7 describes the response of soil and rock to dynamic loading and discusses the effects of past earthquakes, development of velocity profiles, dynamic laboratory tests, and variation of shear modulus and damping with strain. The applicant referred to SSAR Subsection 2.5.2.1 for details regarding the historical earthquake events for the CRN Site.

2.5.4.2.7.1 Velocity Profiles

The applicant conducted various geophysical surveys, including seismic refraction and reflection, and P-S Suspension logging at the CRN Site to characterize in situ dynamic properties of the subsurface materials. The P-S Suspension logging method was used to collect Vs and Vp measurements for each stratigraphic unit and then these unit Vs and Vp profiles were assembled to provide unique Vs and Vp profiles for Locations A and B. The Figures 2.5.4-3 and

2.5.4-4 present geologic and Vs profiles for locations A and B.

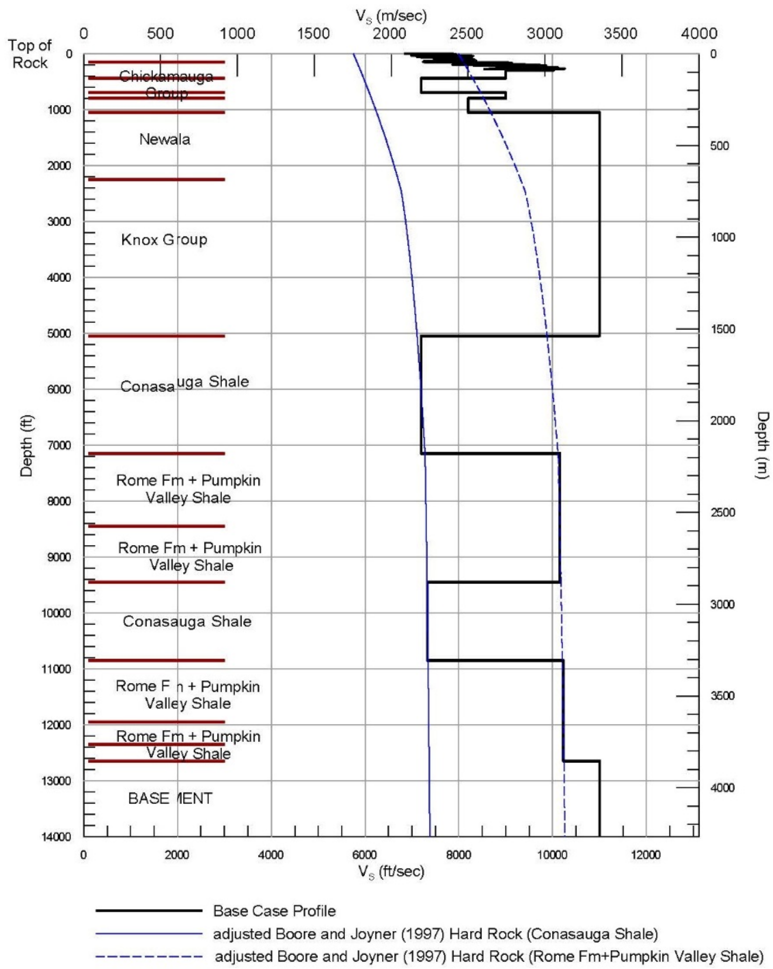

Figure 2.5.4-3: Geologic and Shear Wave Velocity Profile for Location A (Reproduced from SSAR Figure 2.5.4-18)

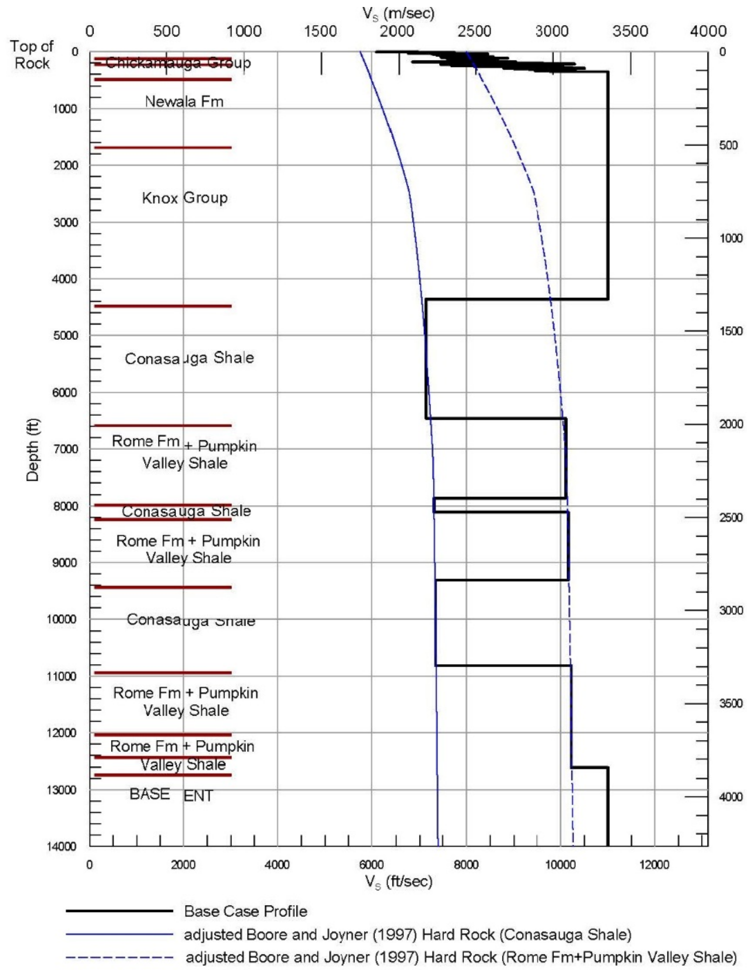

Figure 2.5.4-4: Geologic and Shear Wave Velocity Profile for Location B (Reproduced from SSAR Figure 2.5.4-19)

The applicant stated that the Vs profiles at Locations A and B are developed using site-measured Vs data. The applicant also used data measured in similar nearby geologic units combined with estimated Vs values from literature when no measurements were available. The best estimate (mean) base case Vs profile for the shallow geologic units was developed for each location by computing the lognormal mean profile for the measured Vs data from the boreholes taken within 30.5 m (100 ft) of the location.

Because limited Vs data are available for the deep geologic units at the CRN Site, the applicant used other available information in the development of its Vs profiles. That information included measured data from spectral analysis of surface wave (SASW) surveys in the Conasauga shale, Pumpkin Valley shale, and Rome Formation at the nearby Watts Bar facility. These data were taken from depths of 152.4 and 457.2 m (500 and 1500 ft) and adjusted to the CRN Site. The applicant used generic central and eastern U.S. hard rock Vs profiles for deeper geologic units of the CRN Site. The applicant assigned average Vs values for depths extending into the Newala Formation, below the measured data. Unless supported by measured data, the applicant assigned a Vs value of 3353 m/s (11,000 fps) for the Newala Formation and for the remainder of the Knox Group.

The applicant took the epistemic uncertainty into consideration when developing the Vs profiles. For each location (Locations A and B) the applicant first determined the mean base case (best estimate) Vs profile, and then developed upper- and lower-range base case profiles using a depth-independent scale factor of 1.25, or a plus or minus 25 percent variation about the mean base case profiles. The applicant capped the Vs values for the upper-range base case profiles at about 3505 m/s (11,500 fps). The applicant stated that the uncertainty associated with a scale factor of 1.25 is considered sufficient to account for the potential complexity of seismic wave propagation associated with the dipping stratigraphy at the site. The applicant provided details of those profiles in SSAR Tables 2.5.4-30 and 2.5.4-31 and illustrated them in SSAR Figures 2.5.4-20 and 2.5.4-21. Those figures are reproduced in Figures 2.5.4-5 and 2.5.4-6.

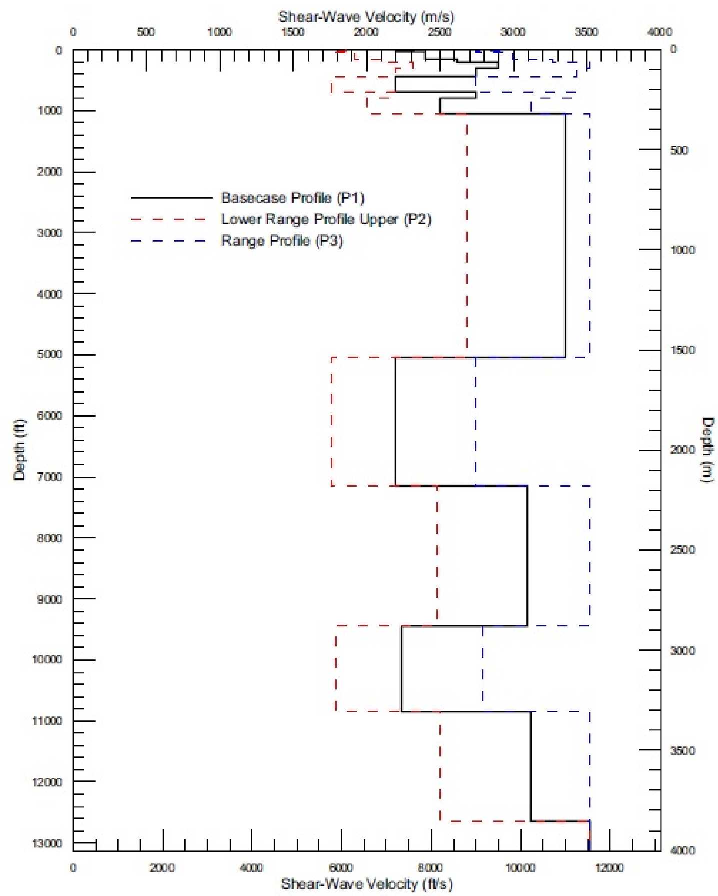

Figure 2.5.4-5: Base case Shear Wave Velocity Profiles for Location A (Reproduced from SSAR Figure 2.5.4-20)

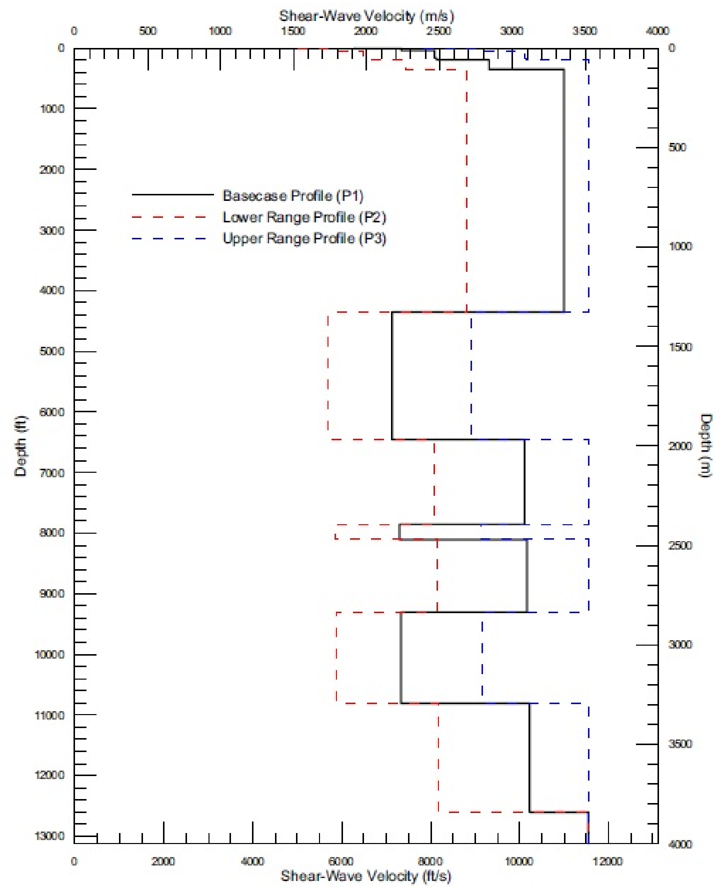

Figure 2.5.4-6: Base case Shear Wave Velocity Profiles for Location B (Reproduced from SSAR Figure 2.5.4-21)

The applicant developed the Vp profiles at Locations A and B in a similar manner as that for the Vs profiles, and illustrated the Vp profiles in SSAR Figure 2.5.4-6.

2.5.4.2.7.2 Dynamic Laboratory Tests

The applicant conducted Resonant Column and Torsional Shear (RCTS) testing in two intact samples of the existing cohesive fill, and compared the test results with the Electric Power Research Institute (EPRI) curves (Plasticity Index [PI] = 30, 40 and 50 percent). The comparison shows that the shear modulus test data aligns reasonably well with the EPRI PI = 30 percent curve, which is supported by the measured PIs of 32 and 33 percent for the test samples. However, the applicant recommends using the EPRI PI = 40 percent curve for both shear modulus reduction and damping because the onsite soils have an overall average PI of 40 percent and the test data reasonably conform to the EPRI curves.

2.5.4.2.7.3 Material Damping and Shear Modulus

The applicant evaluated the dynamic performance of the firm rock material in the upper 152.4 m (500 ft) of the site under linear and nonlinear behavior by using two sets of hysteretic damping and shear modulus reduction curves. A subset of the EPRI rock curves is used to represent the upper-range nonlinearity (M1) in the materials and linear analyses (M2) to represent an equally plausible alternative rock response. The applicant stated that the original depth dependent curves were provided over depths of 15.5 to 36.6 m (51 to 120 ft) and 609.9 to 1524.0 m (2001 to 5000 ft). The curves are modified for the M1 profile to depths of 0 to 6.4 m (0 to 21 ft) and 6.4 to 152.2 m (21 to 500 ft). The applicant further revised the damping curves, reducing the original three percent low strain hysteretic damping to two percent damping. A damping value of 1.25 percent is used to represent a linear response. For rock layers greater than 152 m (500 ft) depth, a linear response is used with a damping adjusted such that the site attenuation (kappa) of the entire profile matches the target kappa. In SSAR Tables 2.5.4-30 and 2.5.4-31, the applicant presented the damping values for the nonlinear (M1) and linear (M2) analyses for each of the best estimate (P1), lower-range (P2) and upper-range (P3) profiles for Locations A and B.

In SSAR Subsection 2.5.4.7.4.2, the applicant described the site attenuation (kappa) specified at the ground surface and zero epicentral distance from the seismic source. The applicant referred to SSAR Subsection 2.5.2.5.1 for additional discussion, and the staff's evaluation of the kappa value is presented in Section 2.5.2.4.5.1 of this report.

2.5.4.2.7.4 Rock Column Amplification/Attenuation Analysis

The rock column amplification/attenuation analysis considers a deep rock profile, from elevation 683 ft to the Precambrian basement rock. The applicant provided a detailed description in SSAR Subsection 2.5.2.5. The applicant referenced SSAR Subsections 2.5.4.7.4 or 2.5.2.5.1 for discussion of the Vs profiles, material damping, shear modulus, and kappa values used in this analysis.

For geologic units above and including the Newala Formation, the applicant used unit weights taken from SSAR Table 2.5.4-21 for its analyses. Unit weights of 26.7 kN/m^3 (170 pcf) for the Conasauga shale, and 27.5 kN/m^3 (175 pcf) for the Pumpkin Valley shale and Rome Formation are assigned to the rock units below the Newala Formation.

2.5.4.2.8 Liquefaction Potential

SSAR Section 2.5.4.8 describes the evaluation of liquefaction potential of the materials adjacent to and under safety-related structures at the CRN Site. The applicant performed geologically-based screening and liquefaction potential analyses in accordance with RG 1.198 "Procedures and Criterua for Assessing Seismic Soil Liquification at Nuclear Power Plants" (NRC 2003 – ADAMS Accession No. ML033280143).

The applicant stated that the safety-related structures at the CRN Site are likely embedded at a depth not to exceed 42.1 m (138 ft) below final grade, and that the sound rock is located about 30.5 m (100 ft) above the foundation level at Locations A and B. If there is any need for repairing the foundation surface, then concrete will be used, therefore there is no potential for liquefaction in the foundation materials. The granular backfill to be used around the structure from top of the rock to finished grade will also not be susceptible to liquefaction because it will be compacted to at least 95 percent of the modified Proctor value.

The applicant assessed the liquefaction potential of existing fill/residual soil at the site. The applicant stated that both the existing fill and residual soils are classified as CH clay. Its liquefaction potential was evaluated in a qualitative manner using criteria for fine grained soils proposed by Polito and Seed et al. Based on the Atterberg limit test results, all of the CRN Site soils fall outside of the proposed zone of liquefiable soils and are not susceptible to liquefaction.

2.5.4.2.9 Earthquake Design Basis

SSAR Section 2.5.4.9 referred to SSAR Section 2.5.2.5.8 for detailed information on the development of the site-specific GMRS.

2.5.4.2.10 Static and Dynamic Stability

SSAR Section 2.5.4.10 describes the evaluation of static and dynamic stability of foundation subsurface materials under safety-related structures. The evaluation includes bearing capacity, heave, settlement, and lateral earth pressures in the power block area at the CRN Site.

The applicant noted that the site is underlain with a succession of stratigraphic units that generally strike N63°E with a dip angle of 33 degrees. The units contain discontinuities, shear-fracture zones, and weathered/fracture zones encountered in the stratigraphic units. The applicant recognized that those discontinuities may impact the stability of foundations and considered the discontinuities when determining rock mass properties and performing foundation stability analyses.

The applicant stated that due to the dipping strata at the CRN Site, the stratigraphic units underlying the power block area vary depending on location. The applicant evaluated two specific locations in the power block area, Location A and Location B, for static and dynamic stability of foundation with embedment of 42.1 m (138 ft) below finished grade (El. 208.2 m (683 ft)).

The applicant used the Hoek-Brown failure criteria (Hoek, E. et al, 2002) and the GSI classification system to determine the rock mass strength. The applicant also used empirical equations and a combination of the intact elastic modulus, GSI, and a disturbance factor (D) in determining bearing capacity, heave, settlement, and lateral earth pressures. The applicant stated that the application of the GSI classification and the Hoek-Brown relationship was based on observation that the rock mass contains several sets of discontinuities that are closely spaced relative to the dimensions of the proposed structure, and that a predetermined failure plane does not exist.

2.5.4.2.10.1 Bearing Capacity

In SSAR Section 2.5.4.10.1, the applicant described the methodologies used for its bearing capacity evaluation. In its evaluation, the applicant considered the limit of the influence zone of loadings under foundation as 134.1 m (440 ft). This value is two times the assumed width of the foundation of 67.1 m (220 ft). This depth of the influence zone is at elevation 116.1 m (381 ft).

At Location A, the Rockdell and Benbolt Formations are encountered at the foundation level with the underlying Fleanor Member within the depth of the influence. This is because of the dipping strata at the CRN Site. At Location B, the Eidson and Fleanor Members are encountered at the foundation level with the underlying Blackford and Newala Formations within the depth of the influence. In its bearing capacity analyses, the applicant separately considered each stratigraphic unit within the depth of influence and treated each unit as a single infinite rock layer below the foundation. The applicant stated that this approach provides a range of bearing capacity values and the most reasonably conservative value is considered.

The applicant examined several methods in its evaluation of rock bearing capacity and noticed that each method generally considers intact rock properties and rock mass properties. The applicant discussed the intact rock properties based on laboratory testing in SSAR Section 2.5.4.2.4.3, and discussed the rock mass properties based on in situ testing in SSAR Section 2.5.4.2.4.4.

Ultimate Bearing Capacity

The applicant used three empirical equations to estimate the ultimate bearing capacity (q_u). Two of these methods, Wyllie (Wyllie, D.C., 1999) and, Kulhawy and Carter (Kulhawy and Carter, 1992) methods, utilize the Hoek-Brown (Hoek. et al, 2002) rock mass constants (m_i, m_b, s, a); and the other method, the U.S. Army Corps of Engineers (USACE) method (USACE, 1994), utilizes a bearing capacity factor based on the friction angle (Φ) of the rock mass.

Allowable Bearing Capacity

Allowable bearing capacity (q_a) is defined as the ultimate bearing capacity divided by a factor of safety (FS). The applicant applied an FS of three to determine the allowable bearing capacities. Those q_a values were computed for the various stratigraphic units within the disturbed (D = 0.7) and undisturbed (D = 0) zones, and for the lower- and upper-bound GSI, underlying Locations A and B. The applicant also used Bowles method (Bowles, J.E., 1992) to estimate q_a of a rock mass by applying a large FS, ranging from 6 to 10 depending on the RQD of the rock. The applicant applied an FS of 6 to estimate q_a when the unconfined compressive strength, σ_{ci} value is available. The applicant used an FS of 6 because all rock units considered in the bearing capacity evaluation have high RQD values, ranging from 80 to 93. SSAR Table 2.5.4-27 provides a summary of the calculated allowable bearing capacity values.

The applicant assumed 431 kPa (9 ksf) for the safety-related foundation load, which is smaller than the existing overburden pressure at the foundation level. Therefore, the net change in pressure at the foundation level, after construction, is expected to be negative. This results in an unloading condition. The applicant stated that the general shear failure, including sliding along a predetermined failure plane, such as a bedding plane, is not likely to occur due to a net decrease in the bearing pressure at the foundation level. Because of the non-general shear failure condition, the applicant stated that the material properties of the rock units (rock mass) are expected to control failure. The applicant also determined that the USACE and Bowles methods are more suited to conditions at the CRN Site. The applicant further stated that the Bowles method does not incorporate GSI or D values, thus there are no associated minimum values. The applicant indicated that the minimum allowable bearing capacity estimates are based on a uniformly disturbed rock mass and considered the estimation overly conservative given the conditions at the CRN Site. Therefore, the applicant recommended that the allowable bearing capacities estimated using the Bowles method be used for design guidance. A rounded low-formation value of 5,266 kPa (110 ksf) is the recommended q_a for the PPE.

For the allowable bearing capacity of concrete, the applicant used ACI standard 318-14 to obtain a q_a value of 7,852 kPa (164 ksf) for lean concrete with 17,237 kPa (2500 psi) strength, which is greater than the recommended q_a for the PPE [5,266 kPa (110 ksf)].

For dynamic bearing capacity, the applicant recommends the same q_a values as that for allowable static bearing capacity, without considering the possible increases of ultimate bearing capacity of rock and concrete fill under dynamic loads that have very short period of loading time.

2.5.4.2.10.2 Settlement and Heave Analysis

In SSAR Section 2.5.4.10.2, the applicant provided analyses for settlement and heave of the foundation at the CRN Site. The applicant used rock mass properties and two methods (Hoek and Diederichs, and Gokceoglu) in its analyses. Similar to the bearing capacity analyses, the applicant considered separately each stratigraphic unit within the depth of influence of a respective foundation as a single infinite rock layer below the foundation.

Settlement Analysis

The applicant stated that the safety-related structures at the CRN Site have an embedment depth not expected to exceed 42.1 m (138 ft) below finished grade; thus these structures would sit directly on bedrock and settlement is expected to be small. Regardless, the applicant estimated settlement for each of the stratigraphic units being considered with an assumed foundation contact pressure of 431 kPa (9 ksf).

The applicant estimated that the total settlements are smaller than 1.27 cm (0.5 in.) for all cases ranging from 0.03 to 0.71 cm (0.01 to 0.28 in.). The applicant provided a summary of the estimated settlements for each of the stratigraphic units in SSAR Table 2.5.4-28.

Heave Analysis

The applicant estimated the total heave due to stress relief during the excavation by using an empirical method suggested by Christian and Carrier (1978) for elastic deformation of an isotropic material. The equation assumes an infinite homogeneous material. The applicant assumed a single infinite rock layer below the foundation for the involved stratigraphic units.

The applicant estimated that the total heave ranges from 0.03 to 0.91 cm (0.01 to 0.36 in.) and that the largest estimated total heave is less than 1.27 cm (0.5 in.). The applicant provided the summary of the heave calculation in SSAR Table 2.5.4-29.

The applicant concluded that the settlement is largely attributed to recompression of the rock. The applicant also concluded that the estimated heave and settlement are expected to be instantaneous, occurring during and shortly after construction. Therefore, the applicant expected no long-term settlement after construction. The applicant pointed out that further analyses of settlement, including differential settlement and heave, need to be performed for the COL application. In addition, the analyses must take into account construction practices, and the specific technology selected, accounting for foundation dimensions, foundation loads, embedment depth, and construction sequence.

2.5.4.2.10.3 Lateral Earth Pressure

SSAR Section 2.5.4.10.3 discusses the methodology to be used in the evaluation of lateral earth pressure exerted on foundation/structure walls below ground.

The applicant suggested the use of Rankine's solution (Bowles, 1988) for determining the static lateral earth pressure with the assumptions that the ground surface behind the top of the wall is flat and there is no friction between the wall and backfill. In addition, the assumptions include internal friction angles of 30 degrees for granular backfill and 20 degrees for in situ soil. The applicant pointed out the need for evaluation of lateral pressures, including hydrostatic pressure, surcharge-induced (equipment and adjacent structures) pressure, and seismic induced pressure. The applicant stated that the evaluation of these components and a full assessment of lateral earth pressure will be performed for the COL application.

2.5.4.2.11 Design Criteria

SSAR Section 2.5.4.11 summarizes the geotechnical design criteria discussed throughout SSAR Section 2.5.4. For evaluation of liquefaction potential, the applicant used the criteria provided in RG 1.198. These criteria specify that cohesive soils, with fines content greater than 30 percent that are either classified as clays or have a PI greater than 30 percent, should generally not be considered susceptible to liquefaction. For its bearing capacity and settlement evaluation, the applicant used settlement limits generally accepted in engineering practices: 15.2 cm (6 in.) for total settlement and 7.6 cm (3 in.) for differential settlement for large mat foundations. For footings, the respective settlement limits are 2.5 cm (1 in.) and 1.3 cm (0.5 in.).

The applicant emphasized that those design criteria and other geotechnical-related criteria related to structural design will be reevaluated or addressed in the COL application and will be specific to the selected technology.

2.5.4.2.12 Techniques to Improve Subsurface Conditions

SSAR Section 2.5.4.12 discusses the soil improvement techniques in the foundation areas of the safety-related structures.

The applicant stated that the impact of karst features on safety-related structures must be evaluated once the locations of these structures have been finalized. The applicant suggested using geophysical subsurface investigation methods to evaluate the presence of karst once the floor of the excavation is reached. The goal of this investigation is to detect any potential voids below the foundation level within a certain zone of influence (void zone of influence). The applicant stated that remediation methods, such as grouting, may be used if anomalies are identified and validated.

The applicant pointed out that it will likely be difficult to obtain a smooth, flat excavation surface due to the dipping stratigraphic units and adjustment of the rock mass. Dental concrete needs to be used, following proper procedures, to create a smooth and level foundation surface.

The applicant stated that an instrumentation plan needs to be developed for the COL application to monitor lateral and vertical displacement during excavation and construction. The applicant suggested installing slope inclinometers and horizontal extensometers to monitor slope movement; installing extensometers to monitor heave in subsurface materials due to the excavation; installing a settlement monitor device to monitor the vertical movement of the foundation; and installing piezometers to monitor changes in pore pressures.

2.5.4.2.13 Foundation Assessment Model

SSAR Section 2.5.4.13 presents a finite element method (FEM) model that was developed to determine potential karstic cavity impacts on SMR foundations. The model can also be used to evaluate bearing capacity and settlement for Locations A and B at 24.4 and 42.1 m (80 and 138 ft) depths at the CRN Site. This FEM model was created using PLAXIS 2D commercial software that is widely used in geotechnical and structural engineering practices.

The FEM model considered Locations A and B with two different cross-sections to account for different rock formations due to the dip of the stratigraphic layers. The model included a disturbed zone around the simulated cavity with appropriate material properties used for cohesion and friction angle. The model also included initial conditions, dewatering assumptions, excavation assumptions, and loading, similar to currently approved new reactor designs.

The FEM simulations examined the foundation stability under various postulated cavity sizes and locations below foundations. The applicant considered the cavity diameters of 1.5 m (5 ft), 3.0 m (10 ft), and 4.6 m (15 ft) based on site investigation boring data. The applicant assumed having infinite length in the horizontal direction; cavity depths of 1.5 m (5 ft) and 9.1 m (30 ft) below foundation embedment depths; foundation embedment depths of 12.2 m (40 ft), 27.4 m (90 ft), and 42.7 m (140 ft); and cavity locations on the edge of the nuclear island, the center of the nuclear island, and on or along bedding planes that were conservatively assumed to feature significant discontinuities or fracture zones.

The results of the FEM model analyses show that the larger cavity has a bigger impact on foundation stability; deeper cavities produce increased relative shear around the cavity; embedment depth does not affect the relative shear force around the cavities but vertical deformation increases for shallower cavity locations; and cavities located on bedding plane discontinuities or in bedding plane fracture zones are most critical and result in the highest shear around the cavity.

Based on the site geologic conditions derived from site investigation data and FEM analysis results, the applicant stated that about 99 percent of the cavities observed in Location A and B borings are significantly less than 3.4 m (11 ft) in height. In addition, cavity development in the CRN Site areas is generally limited to the most markedly weathered zone immediately below ground surface to depths less than 30.5 m (100 ft) and cavity-related failure has a higher potential to occur at depth less than 9.1 m (30 ft) from the ground surface. Given that the foundation embedment depths for SMR designs are much deeper than 9.1 m (30 ft), and that the 4.6 m (15 ft) critical cavity diameter determined by PLAXIS 2D modeling is much greater than that for 99 percent of the cavities observed in CRN Site, the applicant concluded that the proposed Locations A and B are generally suitable for an SMR foundation.

The bearing capacity analysis showed that for Location A, the PLAXIS bearing capacity is 7,037 kPa (147 ksf), compared with 7,133 kPa (149 ksf) using Bowles method. For Location B, the PLAXIS bearing capacity is 5,122 kPa (107 ksf) compared with 5,170 kPa (108 ksf) using Bowles method. The results indicate that the site bearing capacity estimated from the PLAXIS model simulation and using the methods described in SSAR 2.5.4.10 are in reasonable agreement.

The applicant stated that for the COL application, foundation performance will be reevaluated after final technology selection, taking into account specific plant design, specific plant loads, and any potential ground improvement or grouting plans. Final foundation locations will also be reevaluated using specific plant information, with consideration for specific site stratigraphy, subsurface layering orientation, and specific fracture or bedding plane discontinuity zonation.

2.5.4.3 Regulatory Basis

The applicable regulatory requirements for the stability of subsurface materials and foundations are as follows:

- 10 CFR 52.17(a)(1)(vi), as it relates to the requirement for an ESP applicant to prepare an SSAR that contains information on geologic and seismic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, and with sufficient margin for the limited accuracy, quantity and period of time in which the historical data have been accumulated.

- 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," as it relates to the design of nuclear power plant structures, systems, and components important to safety to withstand the effects of earthquakes or deformation.

- 10 CFR 100.23, "Geologic and seismic siting criteria," as it relates to the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and design of nuclear power plants.

- 10 CFR Part 50, Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants," as it relates to the requirements of the quality assurance program to be applied to the design, fabrication, construction, and testing of the structures, systems, and components of the facility.

The related acceptance criteria from NUREG–0800, "Standard Review Plan for the Review of Safety Analysis Report for Nuclear Power Plants (LWR Edition)," Revision 5, dated July 2014, Section 2.5.4 (NRC, 2014 - ADAMS Accession No. ML13311B744) are as follows. Many of these acceptance criteria are not evaluated for an ESP, and are deferred to the COL application. These are indicated within the Technical Evaluation section of this report:

- Geologic Features: In meeting the requirements of 10 CFR Parts 50, 52 and 100, the section defining geologic features is acceptable if the discussions, maps, and profiles of the site stratigraphy, lithology, structural geology, geologic history, and engineering geology are complete and are supported by site investigations that are sufficiently detailed to obtain an unambiguous representation of the geology.

- Properties of Subsurface Materials: In meeting the requirements of 10 CFR Parts 50, 52 and 100, the description of properties of underlying materials is considered acceptable if state-of-the-art methods are used to determine the static and dynamic engineering properties of all foundation soils and rocks in the site area to sufficient depth that impact behavior during construction and over the life of the facility, including during postulated seismic events.

- Foundation Interfaces: In meeting the requirements of 10 CFR Parts 50, 52 and 100, the discussion of the relationship of foundations and underlying materials is acceptable if it includes:

    - (1) a plot plan or plans showing the locations of all site explorations, such as borings, trenches, seismic lines, piezometers, geologic profiles, and excavations with the locations of the safety-related facilities superimposed thereon;

    - (2) profiles illustrating the detailed relationship of the foundations of all seismic Category I and other safety-related facilities to the subsurface materials;

- (3) logs of core borings and test pits; and
- (4) logs and maps of exploratory trenches in the application for a COL.

- Geophysical Surveys: In meeting the requirements of 10 CFR 100.23, the presentation of the dynamic characteristics of soil or rock is acceptable if geophysical investigations have been performed at the site and the results obtained therefrom are presented in detail.

- Excavation and Backfill: In meeting the requirements of 10 CFR Part 50, the presentation of the data concerning excavation, backfill, and earthwork analyses is acceptable if:

  - (1) the sources and quantities of backfill and borrow are identified and are shown to have been adequately investigated by borings, pits, and laboratory property and strength testing (dynamic and static); long-term solubility properties and dissolution behavior during the life of the facility have been determined; and this data is included, interpreted, and summarized;

  - (2) the extent (horizontally and vertically) of all seismic Category I excavations, fills, and slopes are clearly shown on plot plans and profiles;

  - (3) compaction specifications and embankment and foundation designs are justified by field and laboratory tests and analyses to ensure stability and reliable performance over the life of the plant;

  - (4) the impact of compaction methods are incorporated into the structural design of the plant facilities;

  - (5) quality control methods are discussed and the quality assurance program described and referenced;

  - (6) control of groundwater during excavation to preclude degradation of foundation materials and properties is described and referenced. If backfill is to be placed under safety-related structures, proper ITAAC should be specified in the applicant's technical submittal to ensure that the static and dynamic properties of in-place backfill material will be the same as, or better than the design parameters. In case cementitious construction material is to be placed under safety-related structures, proper ITAAC should be specified in the applicant's technical submittal to ensure that the cementitious backfill placed underneath any seismic Category I structures to a thickness greater than 5 ft, meets the design, construction and testing of applicable ACI standards. In addition, the long-term behavior of the backfill subjected to any aggressive groundwater characteristics is evaluated;

- (7) For sites where deeply embedded structures are involved, deep excavation techniques will likely utilize wall retaining systems rather than a sloped excavation of the soil. Also, a description of the planned excavation technique(s) and design of the wall retention system with sufficient details is provided and it should be able to demonstrate that the excavation technique used will not significantly affect the surrounding soil properties that are relied upon in the analysis and design of the foundation and plant structures.

- Groundwater Conditions: In meeting the requirements of 10 CFR Parts 50, 52 and 100, the analysis of groundwater conditions is acceptable if the following are included in this subsection or cross-referenced to the appropriate subsections in SRP Section 2.4 of the applicant's technical submittal:

  - (1) discussion of critical cases of groundwater conditions relative to the foundation settlement and stability of the safety-related facilities of the nuclear power plant;

  - (2) plans for dewatering during construction and the impact of the dewatering on temporary and permanent structures. This includes consideration of the potential for substantial head and volume of water due to the deep excavation for the plant structures;

  - (3) analysis and interpretation of seepage and potential piping conditions during construction;

  - (4) records of field and laboratory permeability tests as well as dewatering-induced settlements;

  - (5) history of groundwater fluctuations as determined by periodic monitoring of an adequate number of local wells and piezometers (flood conditions should also be considered); and

  - (6) evaluation of chemical properties of the groundwater that may impact long-term behavior of the rock/soil/fill materials as well as structural elements (concrete and steel materials).

- Response of Soil and Rock to Dynamic Loading: In meeting the requirements of 10 CFR Parts 50, 52 and 100, descriptions of the response of soil and rock to dynamic loading are acceptable if:

  - (1) an investigation has been conducted and discussed to determine the effects of prior earthquakes on the soils and rocks in the vicinity of the site (evidence of liquefaction and sand cone formation should be included);

  - (2) field seismic surveys (surface refraction and reflection and in-hole and cross-hole seismic explorations) have been accomplished and the data presented and interpreted to develop bounding P and S wave velocity profiles; and

  - (3) dynamic tests have been performed in the laboratory on undisturbed samples of the foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysteretic damping properties of the soils and the results included. If generic soil degradation properties are used in the related preliminary analyses (e.g., site seismic response and SSI analyses), then reconciliation of the generic properties and laboratory testing results should be performed. The section should be cross-referenced with Section 2.5.2.5.

- Liquefaction Potential: In meeting the requirements of 10 CFR Parts 50, 52 and 100, if the foundation materials at the site adjacent to and under seismic Category I structures and facilities are saturated soils and the water table is above bedrock, then an analysis of the liquefaction potential at the site is required.

- Static and Dynamic Stability: In meeting the requirements of 10 CFR Parts 50, 52 and 100, the discussions of static and dynamic analyses are acceptable if the stability of all safety-related facilities has been analyzed from a static and dynamic stability standpoint, including bearing capacity, rebound, settlement, and differential settlements under deadloads of fills and plant facilities; dynamic loads including “live” and seismic loads with consideration of loading sequences and combinations; and lateral loading conditions.

- Design Criteria: In meeting the requirements of 10 CFR Part 50, the discussion of criteria and design methods is acceptable if the criteria used for the design, the design methods employed, and the factors of safety obtained in the design analyses are described and a list of references presented.

- Techniques to Improve Subsurface Conditions: In meeting the requirements of 10 CFR Part 50, the discussion of techniques to improve subsurface conditions is acceptable if plans, summaries of specifications, and methods of quality control are described for all techniques to be used to improve foundation conditions (such as grouting, vibroflotation, bridging mats, dental work, rock bolting, or anchors).

In addition, the geologic characteristics should be consistent with appropriate sections from: RG 1.28, “Quality Assurance Program Requirements (Design and Construction)”; RG 1.27, “Ultimate Heat Sink for Nuclear Power Plants,” RG 1.132, RG 1.138, and RG 1.198, “Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites.”

2.5.4.4 Technical Evaluation

The staff reviewed SSAR Section 2.5.4 to verify that the information contained in the ESP application adequately addresses the required information relating to the stability of subsurface materials and foundations. This section of the SER provides the staff's evaluation of that information. The staff examined the information obtained through geophysical and geotechnical site investigations, which were conducted by the applicant to characterize the geologic conditions and subsurface materials at the CRN Site. The staff examined the applicant's field and laboratory investigation data and the methodologies used to determine the geotechnical properties of the soil and rock underlying the proposed ESP site. The staff also determined if the applicant conducted its site investigations at an appropriate level of detail in compliance with the applicable regulations. The staff evaluated whether the applicant adequately determined the engineering properties of the subsurface materials at the site with consideration of uncertainties and variability. The staff reviewed the subsurface materials and foundation stability analyses, including liquefaction potential assessment, bearing capacity, and settlement estimates under possible static and dynamic (seismic) loadings, with associated assumptions and methods. The staff reviewed the applicant's responses to the staff's requests for additional information (RAI) along with related supplemental information and calculation packages.

On May 8 and 9, 2017, the NRC staff conducted a site audit (NRC, 2017 - ADAMS Accession No. ML17223A428) to examine selected borings, cores, and samples, and to review the geology, seismology and geotechnical modeling and calculation packages. This audit allowed the staff to better understand the actual site conditions including surface and subsurface characteristics of the site; methods and procedures used in determination of soil and rock properties; and evaluations of subsurface material and foundation stabilities. The staff also identified additional information needed to assist its review of the CRN ESP application during this site audit.

2.5.4.4.1 Description of Site Geologic Features

In SSAR Section 2.5.4.1 refers to SSAR Section 2.5.1 for a description of the geologic features at the CRN Site. Section 2.5.1.4 of this report presents the staff's evaluation of the geologic features. The staff reviewed the summary of the description and characterization of the site geology provided in SSAR Section 2.5.4.1 including the site-specific stratigraphy, and foundation stability conditions such as: (1) Stratigraphy; (2) Previous Loading History; and (3) Discontinuities, Shear-Fracture Zones and Weathered Fracture Zones.

Stratigraphy

The staff concentrated its review on the stability of the stratigraphic units within the expected zone of influence at the PPE proposed foundation level of El. 208.2 m (683 ft). The staff focused its review on the impact of the 33 degree inclination of the stratigraphic units at the CRN Site. Due to this inclination, various units may be exposed at the foundation elevation when the excavation is surfaced, making it critical for the staff to ascertain the geometrical components of these units, including the apparent thicknesses of the layers and their true inclination. The staff examined boring logs taken from these locations and was able to map out the interface between layers as described in the applicant's submittal. In addition, the staff reviewed the applicant's supplemental letter dated December 15, 2016 (TVA, 2016 - ADAMS Accession No. ML16350A420), in which the applicant provided a relational analysis between the CRN Site and the former CRBRP site, which are co-located in close proximity, as demonstrated in SSAR Figures 2.5.1-30 and 2.5.1-51. In the aforementioned letter, the information shows that the geologic units mapped in the CRBRP excavation (Fleanor Member and Rockdell Formation) are the same units as those occurring in Location B of the CRN Site and have a similar inclination. During the May 2017 site audit, the staff reviewed CRN Site deep borings, MP 101 and MP 102, obtained from location A of the CRN Site. Based on visual inspections, the staff was able to correlate the actual boundaries between geologic layers with the applicant's descriptions. Based on the aforementioned data, the staff concludes that the applicant adequately described the stratigraphy at the CRN Site.

Previous Loading History

The staff reviewed the references related to previous loading history at the CRN Site provided in SSAR Section 2.5.4.1. The staff reviewed SSAR Figure 2.5.4.-1 and SSAR Table 2.5.4-2 where the applicant presented differences between the historic ground surface elevations prior to site development for the CRBRP subsurface investigation. The staff agrees with the applicant's assessment of loading history at the site, based on up to 6.1 m (20 ft) of fill placed in the southern portion of the power block area, and up to 21.3 m (70 ft) of material removed from the central and northern portions.

Discontinuities, Shear-Fracture Zones and Weathered Fracture Zones.

The applicant referred to SSAR Section 2.5.1.2.4 for a detailed discussion of discontinuities, shear-fracture zones, and weathered fracture zones at the CRN Site. The applicant described the shear-fracture zones as closely spaced, tightly healed, calcite filled shear fractures with an average apparent thickness of about 1.2 m (4 ft). The staff reviewed SSAR Figures 2.5.1-65 through 2.5.1-67, which show a set of cross-sections through all shear-fracture zone features encountered from borings drilled for the CRBRP and CRN Site subsurface investigations. The staff noted that a shear zone was reported to have been encountered during the subsurface investigation for the CRBRP in the lower portion of the Eidson Member. Similarly, the staff noted that a shear-fracture zone was identified within the Eidson Member for the CRN Site investigation. In addition, the staff noted that the applicant identified, in SSAR Figure 2.5.1-67, a shear-fracture zone in the Rockdell and Benbolt Formations. The staff noted that both shear fractures zones range in elevations between 228.6 and 137.2 m (750 and 450 ft). These representations provide evidence of shear-fracture zones at or close to the expected deepest foundation level of 208.2 m (683 ft). The applicant stated that detailed geologic mapping will be performed during excavation for the power block area to further characterize any shear-fracture zones encountered. As documented in TVA letter dated December 15, 2016 (TVA, 2016 - ADAMS Accession No. ML16350A420), the applicant committed to perform a detailed geologic mapping of excavation walls during excavation and construction, allowing documentation of the characteristics of dissolution features in the near-surface carbonate rock units and verification of the decrease in cavity size and abundance with depth. Section 2.5.3.5, "Geologic Mapping Permit Condition," of this report identifies Permit Condition 1 as the COL or CP applicant's responsibility to perform detailed geologic mapping of excavations for safety-related engineered structures at the CRN Site.

The staff reviewed SSAR Table 2.5.1-16 and noted that 52 fracture zones were listed with ten of those zones occurring below the power block area at an elevation of about 208.2 m (683 ft) NAVD 88. In SSAR Section 2.5.4.3, the applicant stated that further evaluation of the shear-fracture zones and weathered fracture zones will be performed in support of the COL application, when the reactor technology is selected. Given that the site investigation data indicate that bedding fractures have weathering or weakening below power block foundation level, and consistent with the applicant's stated intentions, the staff identified the following COL action item:

COL Action Item 2.5-1

> An applicant for a COL or CP referencing this early site permit, upon selection of a final technology and site location, should conduct further evaluation of the shear-fracture zones and weathered fracture zones at the CRN Site.

Similar to the shear-fracture zone review, the staff independently reviewed boring logs and information in SSAR Table 2.5.1-16 to ascertain the applicant's descriptions of the weathered fracture zones. The staff reviewed Appendix B of the Geotechnical and Exploration and Testing Data Report (AMEC, 2014) and noted that most borings drilled at the CRN Site indicate the presence of weathered or fracture zones within the stratigraphic units. In accordance with the applicant's descriptions, these zones typically represent poor to fair quality rock consisting of multiple, healed to open, slightly to highly weathered fractures or bedding planes, some calcite or dolomite filled, with occasional core loss and loss of drilling fluid reported, and with apparent thickness of 0.9 m (3 ft).

In SSAR Section 2.5.1.2.6.3, the applicant indicated that the uppermost weathered zone is at a depth of 30.5 m (100 ft) or less from the surface, and that rock mass discontinuities become tighter, less frequent, and shorter as depth increases. The applicant described the condition of the joints as undulating to planar, rough to smooth to slickensided, very tight to open with tightly healed to slightly altered joint walls, and partially or wholly filled with calcite. The staff reviewed actual cores from selected borings during the site audit, and observed discontinuities within the rock units. The staff asked the applicant, in eRAI 9035 (RAI no. 6), Question 2.5.4-1, to discuss how the inclined rock formation interfaces were taken into account when determining the rock mass properties. The staff evaluated some of the information from the applicant's RAI response in the following text. The staff provided some additional evaluation of this RAI response later in SER Section 2.5.4.4.2.

As part of the response to eRAI 9035 (RAI no. 6), Question 2.5.4-1, the applicant used geotechnical coring logs, rock core photographs, acoustic televiewer logs, and information on shear-fracture zones to characterize rock mass discontinuities and fracture zones. The applicant performed an assessment for bedding fractures and joints in 15 borings and found a total of 1,997 bedding joints and associated fracture zones ranging in depth from 18.4 to 163.9 m (60.4 ft to 537.8 ft). The applicant indicated that for the bedding joints at drilled depth greater than 30.5 m (100 ft) only 57 bedding joints and associated fracture zones have non-softening clay coatings to softening clay fillings less than 5 millimeters (mm) (0.2 ft) in thickness. In addition, the applicant performed an assessment based on acoustic televiewer logs on 14 borings and identified 2,438 bedding structures ranging in depth from 2.6 to 163.7 m (8.4 to 537.1 ft) and a total of 860 fractures ranging in depth from 1.9 to 163.5 m (6.2 to 536.4 ft). The applicant indicated that for fracture zones at drilled depth greater than 30.5 m (100 ft), only 1.7 percent of the fracture zones were identified as open, planar, and with similar orientation to the average bedding orientation.

The staff reviewed the applicant's response to eRAI 9035 (RAI no. 6), Question 2.5.4-1 (TVA, 2017 – ADAMS Accession No. ML17261A062), and noted that the applicant's assessment of discontinuities, and weathered and shear-fracture zones, was primarily focused on results below 30.5 m (100 ft). The staff noted that observations from the CRBRP excavations are consistent with the CRN Site investigation data regarding depth of weathering and improvement in rock mass discontinuity conditions below a depth of 30.5 m (100 ft). While reviewing the CRBRP observations from construction excavation, the staff noted that the excavation for the CRBRP nuclear island was about 30.5 m (100 ft) deep. The staff reviewed the Acoustic Televiewer (ATV) data provided in Appendix C of the Geotechnical and Exploration and Testing Data Report (AMEC, 2014) and was able to confirm the applicant's assertion that most discontinuities occur along bedding planes and within weathered rock in the upper 30.5 m (100 ft) of bedrock.

The applicant stated that the effect of shear-fracture zones and weathered fracture zones were incorporated into the average GSI for each stratigraphic unit at the site for rock mass characterization. The staff noted that in SSAR Section 2.5.1.2.6.2, the applicant indicated that the GSI classification, which accounts for the effect of weathered fracture zones and shear-fracture zones for rock mass characterization, is applied to bedrock stratigraphic units below El. 225.9 m (741 ft) NAVD88. The applicant provided detailed information about rock mass properties in the stratigraphic units in SSAR Section 2.5.4.2.4.4, including how it accounted for discontinuities, weathered and fracture zones, and shear-fracture zones. The staff's evaluation of the use of the GSI method for rock mass characterization is discussed in Section 2.5.4.4.2 of this SER.

In SSAR Section 2.5.4.1.3.3, the applicant stated that weathered and fracture zones typically occur along bedding planes or fractures and likely represent early dissolution of the limestone. The weathered and fracture zones are characterized as zones of poor to fair quality rock with slightly to highly weathered fractures or bedding planes. In SSAR Section 2.5.1.2.6.1, the applicant provided a brief summary of the subsurface conditions and indicated that the estimated shallowest foundation level within the power block area is approximately 24.4m (80 ft) below the final grade at El. 225.9 m (741 ft) NAVD88, and the deepest foundation level is not expected to exceed a depth of approximately 42.1 m (138 ft) below final grade at El. 208.2 m (683 ft) NAVD88. The staff also noted that most of the discontinuities, weathered and fracture zones will be excavated during the construction process which will generally minimize the adverse effects of those zones on the stability of foundations and structures.

The staff identified Permit Condition 2 because the discontinuities, shear fractures zones, and weathered fracture zones typically exist along bedding planes and within weathered rock in the uppermost 30.5 m (100 ft), where most of the cavities are encountered at the CRN Site. In addition, the staff noticed that the applicant's rock mass characterization is mainly for bedrock stratigraphic units below 24.4 m (80 ft) (El. 225.9 m (741 ft) NAVD88), which is defined as the estimated shallowest foundation level. Permit Condition 2 addresses the requirement of excavation for the upper 24.4 m (80 ft) of the materials in safety-related structure areas in order to minimize the adverse effects of discontinuities, weathered and shear-fracture zones, and karst features on the stability of subsurface materials and foundations. This Permit Condition is described in Section 2.5.4.5 of this report.

Karst Features

The applicant referred to SSAR Section 2.5.1.2.5 for a detailed analysis of karst features at the CRN Site. The staff focused its review on the applicant's PLAXIS 2D finite element (FE) model described in its supplemental letter dated July 3, 2017 (TVA, 2017 - ADAMS Accession No. ML17186A113), and the applicant's description of karst features. The staff reviewed Appendix D of the applicant's supplemental letter which includes a table that summarizes all cavities observed in the CRN Site borings. The staff noted that a total of 233 cavities were reported by the applicant, among which 26 cavities are below elevation 225.6 m (740 ft) (shallowest embedment structure foundation depth considered), and the biggest cavity, with a height of 2.9 m (9.5 ft), was found at a median elevation of 224.1 m (735.4 ft) in the Eidson Member.

The staff reviewed multiple boring logs within the power block area, including field notes and descriptions of these features. The staff paid particular attention to boring log MP-424, which shows the presence of considerably large cavities close to the deepest foundation level. Boring MP-424 shows four cavities within approximately 6.1 m (20 ft) of the ground at elevation 208.2 m

(683 ft) and with maximum height of 1.3 m (4.3 ft). In addition, the staff reviewed SSAR Table 2.5.1-11, which shows all identified cavities in boreholes drilled at the site for the CRBRP investigation (1973-1978) and for the current CRN investigation (2013). The staff noted that cavities are present in each of the stratigraphic units at the site with heights ranging from less than 0.3 m (1 ft) to approximately 5.2 m (17 ft). The staff also reviewed SSAR Figures 2.5.1-51 and 2.5.1-52, which show the spatial distribution of the karst features at the site. The staff noted that most of the cavities occur within approximately 30.5 m (100 ft) of the ground surface, and are located mostly in the Rockdell Formation and the Eidson Member, which are the two strata located next to the foundation strata for CRN Locations A and B.

The applicant developed a FE model using PLAXIS 2D to evaluate the potential impacts of karstic cavities on SMR foundations. The staff's review of this analysis focused on assessing the suitability of the site as it relates to the critical size of a cavity that can affect foundation stability. The applicant modeled a foundation using a typical nuclear power plant layout to determine the analysis input parameters, including foundation dimension, thickness, loading, and deformation limits. In addition, the applicant accounted for varying dip of the stratigraphic layers and included a disturbed zone around the simulated cavity with appropriate material properties. The applicant used, for the nuclear island basemat area, a final plant grade at El. 250.2 m (821 ft) NAVD88 and considered multiple embedment depths (12.2 m (40 ft), 27.4 m (90 ft) and 42.7m (140 ft)) below the ground surface). The applicant evaluated three cavity sizes, 1.5 m (5 ft), 3.1 m (10 ft), and 4.6 m (15 ft), and two different cavity depths below the foundation level (1.5 m (5 ft) and 9.1 m (30 ft)) on multiple cavity locations (edge of the NI, center of the NI, and along bedding planes) for site locations A and B under static loading conditions. The staff noted that selected cavity sizes are similar to those observed during field investigations, and the postulated locations of cavities along bedding planes replicated the presence of shear-fracture zones. The applicant evaluated the potential collapse of cavities in terms of relative shear. The applicant selected a critical relative shear of 0.85 (85 percent of the material strength) to provide a margin of safety. The staff reviewed SSAR Table 2.5.4-34, which includes the model analysis results for Locations A and B. The staff noted that for all evaluated foundation depths, the analysis results indicate that the most critical condition is a postulated cavity with a diameter of 4.6 m (15 ft) located either on a bedding plane discontinuity or in a bedding plane shear-fracture zone. Based on its review of the FE model, the staff concludes that the applicant conducted an appropriate preliminary evaluation to determine potential karstic cavity impacts on SMR foundations. This analysis should be site and technology specific. Therefore, A COL or CP applicant referencing the CRN ESP, should consider, at a minimum, the following: specific plant design, loads, ground improvement, grouting plans, final foundation location, site stratigraphy, subsurface layering orientation, and specific shear failure or bedding plane discontinuity zones. The staff identified COL Action Item 2.5-2 for the reevaluation of foundation performance upon selection of a final technology and site location.

COL Action Item 2.5-2

> Upon selection of a final technology and site location, an applicant for a COL or CP referencing this early site permit, should reevaluate the potential of karstic cavity impacts, within the zone of influence of the foundation under all design loading conditions, on foundation stabilities for safety-related structures. The evaluation should be performed using a method that can adequately model foundation performance under actual site geologic conditions and specific loading conditions. In the evaluation, detailed

information should be provided to address the site subsurface geologic characteristics, foundation dimension and embedment depth, the lateral location of the foundation with respect to the bedding planes and shear-fracture zones, location and dimension of voids, the shear strength at the bedding planes and shear-fracture zones, the in situ stresses around the foundations, and proper subsurface material properties to be used. The analysis should also take into account undetected cavities that could adversely affect foundation performance and include details related to the expected size of such a potential cavity.

The staff reviewed supplemental information to the application by letter dated December 15, 2016, (TVA, 2016 - ADAMS Accession No. ML16350A420), which presented a comparison of the CRN Site with the CRBRP site with respect to geologic formation, rock type, geologic structure, and character of karst and voids/cavities encountered at and below foundation depths. The staff also reviewed SSAR Figures 2.5.1-75 through 2.5.1-77, which depict the average elevation of cavity and cavity size from the CRN and CRBRP site investigations. The staff noted that for both site investigations, the majority of the cavities occur within approximately 30.5 m (100 ft) of the ground surface. The staff compared SSAR Figures 2.5.1-50 and 2.5.1-37, which show the geologic mapping for the CRBRP and CRN Site investigations, respectively. The staff noted that karst-related features include large funnel-shaped and dish-shaped sinkholes, and that both site investigations identified small holes in the ground. The staff also noted that the applicant-mapped surface features for the CRN Site using high-resolution LiDAR topographic data, and identified the same two major sinkhole clusters, along with several additional sinkholes.

In the same supplemental information, and in SSAR Section 2.5.1.2.6.10, the applicant provided a mitigation plan outlining additional actions to be detailed in the COL application. These actions will be completed as a part of the site excavation and construction to confirm the current understanding of karst features at the CRN Site; to ensure that the size, distribution, and extent of karst cavities are sufficiently understood; and to understand the potential impact of the cavities on safety-related structures. The applicant's mitigation plan includes detailed geologic mapping of the excavation floor, development of a grouting program; and additional geophysical surveys to detect and address possible cavities at and below the foundation levels. The staff agrees that this plan is needed in order to fully map out and assess the presence of karst features, including open or filled cavities, at or below the expected foundation depths. Section 2.5.3.5 of this report identifies Permit Condition 1 as the COL or CP applicant's responsibility to perform detailed geologic mapping of excavations for safety-related engineered structures at the CRN Site. Consistent with applicant's statements in SSAR Section 2.5.1.2.6.10 and the applicant's supplemental information, the staff identified the following COL action item:

COL Action Item 2.5-3

> An applicant for a COL or CP referencing this early site permit should design and conduct additional surface geophysical surveys during excavation and construction to detect cavities below the foundation elevation that could adversely affect foundation performance. In addition, the applicant should perform confirmatory drilling or borehole testing during excavation/construction to characterize the source of geophysical anomalies, and should develop a grouting program with associated ITAAC when needed, based on the information obtained by the geologic mapping, geophysical surveys, and specific analyses, to mitigate the effect of voids or cavities on foundation performance at and below the foundation levels of safety-related structures.

Based on its review, the NRC staff concludes that the applicant provided sufficient information to characterize karst features at or below the expected foundation elevation at the CRN Site for the ESP. In addition, the staff finds that the applicant conducted an appropriate preliminary evaluation to determine potential karstic cavity impacts on SMR foundations. The staff acknowledges that more detailed information regarding the presence of karst features at or below the expected foundation level needs to be provided in a COL or CP application.

Unrelieved Stresses in Bedrock

The applicant refers to SSAR Section 2.5.1.2.6 for a complete discussion of unrelieved stresses in the bedrock at the CRN Site. The staff focused its review on the applicant's description of unrelieved stresses in the bedrock at close proximity to the expected foundation elevation of 208.2 m (683 ft). The staff noted that during the expected excavation process, removal of overburden may cause current discontinuities close to the surface of the excavation to open and may also create new discontinuities due to the release of overburden stress. The staff acknowledges that blasting techniques would likely be used during the excavation process that may also cause additional discontinuities within the bedrock especially adjacent to the foundation. The staff noted that the applicant considered this disturbed zone as part of the rock mass properties characterization. The staff evaluated the rock mass properties characterization in Section 2.5.4.4.2 of this report.

Conclusions Regarding Site Geological Features

Based on the review of SSAR Section 2.5.4.1, the staff concludes that the applicant provided sufficient information to characterize geologic features at or below the expected foundation elevation for the current ESP, and meets the requirements of 10 CFR 100.23. In addition, the staff proposed Permit Conditions 1 and 2, as described in SER Sections 2.5.3.5 and 2.5.4.5, respectively, to be imposed on this ESP to address geohazard-related safety matters; and as described in COL Action items 2.5-1 through 2.5-3, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.

2.5.4.4.2 Properties of Subsurface Materials

The staff focused its review of SSAR Section 2.5.4.2 on the applicant's description of the static and dynamic engineering properties of the CRN Site subsurface materials, and the methods used to determine the engineering properties. The staff reviewed the applicant's field investigation methods and the laboratory testing program as well as the assumptions used to determine the engineering properties. The staff performed its review in accordance with the guidance in RG 1.132, RG 1.138, RG 1.208 and NUREG–0800, Section 2.5.4.

Description of Subsurface Materials

The staff reviewed the information provided in SSAR Section 2.5.4.2.1.1 and SSAR Table 2.5.4-3, which shows depths and thicknesses of existing fill and residual soil encountered in the borings at the CRN Site. The staff reviewed borings logs obtained from the site subsurface investigation and noted consistency for the depths and thicknesses shown on the aforementioned table. The staff noted that the applicant used standard industry methods to describe properties of the fill. The applicant classified the soil as a CH clay based on the USCS, ASTM D2487. Based on its review, the staff agrees with the applicant's classification of existing fill and residual soils and the applicant's assessment of fill and soil thickness.

The applicant stated that the backfill surrounding the safety-related structures consists of both lean concrete and granular backfill with lean concrete extending from the foundation level to the top of the rock, and granular backfill extending from the top of rock to the finished grade. The applicant provided a detailed description of the proposed backfill material in SSAR Section 2.5.4.5. The staff's evaluation of backfill is provided in Section 2.5.4.4.5 of this report.

In SSAR Section 2.5.4.2.1.3, the applicant described the information related to weathered rock that was encountered in most of the borings drilled at the site. The staff reviewed information provided by the applicant to support its description of weathered rock at the CRN Site, including RQD values, V_s values, and rock core photographs. The applicant reviewed RQD values in accordance with ASTM D6032 standard, to further define the thickness of weathered rock and the corresponding depth to the bedrock. The applicant used RQD values of equal to or less than 25 percent to represent very poor quality rock, and V_s values that are significantly lower than average shear wave velocity value of the same rock formation, to define the zone of the weathered rock. The applicant determined the maximum thickness of weathered rock at the CRN Site as approximately 11.9 m (39 ft). The staff noted that the applicant used state-of-the-art methods to properly characterize the extent of weathered rock at the CRN Site. The applicant indicated that weathered rock would be excavated prior to the construction of foundation. To ensure that safety-related structures will be founded on competent subsurface materials, the staff identified the following COL action item:

COL Action Item 2.5-4

> An applicant for a COL or CP referencing this early site permit will need to excavate all weathered rock from safety-related structure areas prior to the construction of foundations at the CRN Site.

The staff reviewed the subsurface profiles and materials described in SSAR sections 2.5.4.2.1.3 through 2.5.4.2.1.11. The staff reviewed SSAR Figure 2.5.4-2, which shows the bedrock structure and succession of the stratigraphic units underneath the power block area. The stratigraphic units encountered at the site include: Benbolt Formation, Rockdell Formation, Fleanor Member (Lincolnshire Formation), Eidson Member (Lincolnshire Formation), Blackford Formation and Newala Formation. SER Table 2.5.4-1, summarizes the characteristics and properties of these formations. The staff focused its review on the Benbolt Formation and the Fleanor Member of the Lincolnshire Formation, which are the stratigraphic units on which the power block of the nuclear power plant is expected to be founded. The applicant described the Benbolt Formation as a gray limestone that is strong, very thinly bedded and locally moderately bedded. The applicant also described the Benbolt Formation as a nodular limestone with little to some laminated to thinly bedded calcacerous siltstone. The staff noted that the average vertical thickness of the formation is approximately 100.6 m (330 ft) and that the RQD averages 88 percent, which is indicative of a good quality rock. The applicant described the Fleanor Member as a red bedrock that is medium strong, to strongly laminated, to medium bedded calcacerous siltstone with few to little gray micritic limestone layers. The staff noted that the average vertical thickness of the formation is approximately 78.3 m (257 ft) and that the RQD averages 89 percent, which is indicative of a good quality rock. The staff noted that the applicant used standard industry methods and techniques to describe the stratigraphic units at the site. The staff reviewed boring logs, field notes, and photographs to verify the applicant's descriptions of the stratigraphic units at the site. Based on its review of the information provided, the staff determines that the applicant appropriately described the stratigraphic units at the CRN Site.

Field Investigations

The applicant referred to SSAR Section 2.5.4.3 and 2.5.4.4 for details on field investigations and geophysical surveys performed at the CRN Site. The applicant indicated that the field investigation was performed in accordance with RG 1.132. The staff's evaluation of the applicant's field investigations and geophysical surveys performed for the CRN Site is presented in Sections 2.5.4.3 and 2.5.4.4.4 of this report.

Laboratory Testing

The staff reviewed SSAR Section 2.5.4.2.3 related to the laboratory testing program performed by the applicant to identify, classify, and evaluate the physical and engineering properties of the soil and rock at the CRN Site. The applicant conducted the laboratory testing program in accordance with an approved quality assurance program following the guidance presented in RG 1.138. The staff reviewed SSAR Table 2.5.4-7, which lists the types and numbers of tests completed for the CRN Site subsurface investigation. The staff reviewed the test results for soil and rock core samples described in the SSAR. The staff noted that the applicant used soil samples to obtain basic characteristics such as grain size, natural moisture content and plasticity, and the shear strength and compaction characteristics. The applicant used rock samples to obtain the unit weight, specific gravity, compressive strength, shear and elastic moduli, Poisson's ratio, slake durability, and calcium carbonate content. The staff reviewed Appendices F through H of the Geotechnical and Exploration and Testing Data Report (AMEC, 2014), which contains details and references for the industry standards used for the testing. The staff concludes that the applicant used proper standard industry methods and performed a wide array of laboratory tests to determine soil and rock properties for the CRN Site in appropriate detail to satisfy its review of the CRN Site ESP application.

Engineering Properties

The staff reviewed SSAR Section 2.5.4.2.4, which presents the engineering properties of the soil, weathered rock, intact rock, and rock mass. The staff focused its evaluation on the methods used to develop the properties which are derived from the subsurface investigation and laboratory testing program. The staff reviewed SSAR Table 2.5.4-21, which contains the selected engineering property values for the subsurface materials at the CRN Site.

<u>Soil Properties</u>

The applicant developed soil properties based on the results of its field and laboratory testing programs. The staff reviewed the summary of the field measured N values presented in SSAR Table 2.5.4-8 and the recommended SPT N_{60} presented in SSAR Table 2.5.4-21. The staff noted that for the SPT N_{60} recommended values, the applicant corrected the field values for factors of energy, boring diameter, and sampler type and rod length. The staff reviewed a summary of the results of the sieve analyses, natural moisture contents, and Atterberg limits performed on fill and residual soil samples. The staff agrees with the applicant classification of both the existing fill and residual soils as high plastic clays. The applicant used methods such as unconsolidated triaxial testing and empirical relationships to characterize the undrained shear strength. The applicant determined the drained shear strength, effective cohesion, and internal friction angle from the CU triaxial testing. The applicant used P-S velocity methods for the shear and compression wave velocity measurements and calculated Poisson ratio and shear modulus. The staff reviewed all information related to the characterization of the geotechnical engineering properties of the soils presented in SSAR Table 2.5.4-21, and concludes that the applicant provided representative values for the soils at the CRN Site. The staff concludes that the applicant used appropriate standard industry methods and tests to characterize the soil properties at the CRN Site and to satisfy its review of the CRN Site ESP application.

Weathered Rock Properties

The staff reviewed SSAR Section 2.5.4.2.4.2, which describes the weathered rock properties at the CRN Site. The applicant developed the engineering properties of the weathered rock for use in the site seismic response analysis. The staff noted that the applicant developed the properties from in situ testing and material correlations. The staff reviewed Appendix C of the Geotechnical and Exploration and Testing Data Report (AMEC, 2014) and noted that a limited number of V_s and V_p measurements were taken in the weathered rock as part of the field testing program. The staff acknowledge that the applicant will excavate all weathered rock during construction, and identified COL Action Item 2.5-4 in Section 2.5.4 4.2 of this report as the responsibility of COL or CP applicant.

Intact Rock Properties

The staff reviewed SSAR Section 2.5.4.2.4.3, which contains the intact rock properties developed for the stratigraphic units encountered underneath the power block areas. The applicant developed the properties based on the results of the laboratory and field testing programs performed at the CRN Site. The staff reviewed SSAR Table 2.5.4-21, which contains the selected values of the engineering properties that characterize the rock formations of the Bentbolt, Rockdell, Blackford and Newala Formations and the Fleanor and Eidson Members.

The staff reviewed the unconfined compressive strength for each formation, which is considered one of the most important intact rock properties for foundation design. The applicant presented a summary of the unconfined compression test results in SSAR Table 2.5.4-15. The staff noted that the average unconfined compressive strength values of 31 to 137.9 MPa (4500 to 20,000 psi) for all rock formations are reasonable, and indicative of intact rock behavior. The staff reviewed SSAR Figures 2.5.4-5 and 2.5.4-6 ,which plot the Vs and Vp measurements for each stratigraphic unit against depth, and noted that the test values are typical for the rock formations. The staff reviewed SSAR Table 2.5.4-19, which provides the slake durability index test results as a measure of susceptibility of rock to slacking. The staff noted high durability indices, above 94 percent on average, for the Benbolt, Rockdell, Blackford Formations and for the Fleanor Member. The staff reviewed the applicant's calcium carbonate content test results presented in SSAR Table 2.5.4-20. The staff noted that the Benbolt Formation had a 27 percent calcite equivalent but only one test was performed. The applicant performed five tests on the Fleanor Member resulting in an average of 34 percent of calcite equivalent. The Rockdell Formation and Eidson Member tests resulted in more than 53 percent of calcite equivalent, on average. For the Newala Formation, the staff noted 84 percent of calcite equivalent on average, with only two tests performed on this formation. The applicant performed one test on the Blackford Formation with a result of 39 percent of calcium equivalent. The applicant also performed pressuremeter testing and provided the shear moduli at various level of strain. The staff noted that results are indicative of a strain hardening behavior. The staff acknowledge that the applicant used appropriate standard industry methods and tests to characterize the intact rock properties at the CRN Site and to satisfy its review of the ESP application.

Rock Mass Properties

The staff reviewed SSAR Section 2.5.4.2.4.4 and focused its review on the applicant's description of the rock mass properties. The applicant accounted for discontinuities and other features when developing the rock mass properties of the stratigraphic units. In SSAR Section 2.5.4.2.4.4, the applicant stated that the rock mass properties are developed using the GSI classifications of the stratigraphic units. The site investigation data indicates the presence of rock discontinuities and fractures in the stratigraphic units, and that the weathered or fracture zones typically occur along bedding planes at the CRN Site. The discontinuities and fracture zones may result in predetermined shear failure surfaces. Because the GSI chart may not be applicable when structural planes of inclined rock surfaces control the failure of a rock mass, the staff asked the applicant, in eRAI-9035 (RAI No. 6), Question 02.05.04-01 (NRC, 2017 - ADAMS Accession No. ML17213A957), to discuss how the inclined rock formation interfaces were taken into account when determining the rock mass properties.

The applicant provided a response to eRAI-9035 (RAI No. 6), Question 02.05.04-01, in Response Letter CNL-17-099 dated September 15, 2017 (TVA, 2017 - ADAMS Accession No. ML17261A062). In its response, the applicant stated that the use of the GSI classification system is applicable to rock masses with many joint sets that create interlocking blocks of rock and where the block sizes are small relative to the length of the potential failure surface. Therefore, the applicant concluded that use of the GSI classification system is applicable to CRN Site. The applicant combined the inspection of rock cores, interpretation of downhole geophysical survey data, and observations made during field mapping for the CRBRP site to characterize the discontinuities of the rock units and rock mass properties of inclined rock units, and to determine whether the presence of continuous weathered or fracture zones could provide a predetermined failure plane. The applicant concluded that weathered and fracture zones are mostly encountered in the uppermost 30.5 m (100 ft), and that the rock mass below 30.5 m (100 ft) is typically tighter and contains less frequent and less persistent discontinuities, and therefore do not result in predetermined failure surfaces. In addition, the applicant stated that this conclusion is supported by its observation from the grouting program conducted at the CRBRP site.

The staff evaluated the applicant's response to eRAI-9035 (RAI No. 6), Question 02.05.04-01. As documented in Section 2.5.4.4.1 of this report, the staff reviewed the applicant's evaluation of weathered and fracture zones. The staff reviewed boring logs and SSAR Table 2.5.1-16 to confirm the applicant's descriptions of the weathered fracture zones. The staff reviewed Acoustic Televiewer (ATV) data provided in Appendix C of the Geotechnical and Exploration and Testing Data Report (AMEC, 2014) and was able to confirm the applicant's assertion that most discontinuities occur along bedding planes and within weathered rock in the upper 30.5m (100 ft) of bedrock. Therefore, there is no sufficient evidence to support the existence of predetermined failure surfaces below the proposed CRN Site foundation level. The staff reviewed "Quantification of the Geological Strength Index Chart," (Hoek, Carter, Diederichs, 2013) to evaluate the applicability of the GSI classification system used for estimating the mechanical properties of the rock masses at the CRN Site. This reference indicates that one important assumption for the GSI classification system is that if a site contains several discontinuity sets that are sufficiently closely spaced, relative to the size of the structure under consideration, then the rock mass can be considered homogenous and isotropic. The staff acknowledged the presence of many joint sets and discontinuities at the CRN Site, and that the weathered and fracture zones are typically tighter and contain less frequent and less persistent discontinuities below 30.5 m (100 ft) from the ground. The staff also notes the applicant's assertion that the power block area excavation is expected to be much larger than the rock blocks that make up the rock mass at the site. Based on its review of the applicant's RAI response, the staff determines that the GSI classification system is applicable for the estimation of the rock mass properties at the CRN Site. Accordingly, the staff considers eRAI-9035 (RAI No. 6), Question 02.05.04-01 resolved.

The applicant developed the rock mass strength using the generalized Hoek-Brown failure criterion available in the computer program RocData. The staff acknowledges that the GSI is a key input parameter in the Hoek-Brown failure criterion, and reviewed SSAR Table 2.5.1-15, which contains the summary of the GSI parameter for each stratigraphic unit. The staff noted the applicant's recommended GSI range values of 70 to 80 percent for the Benbolt Formation and 65 to 85 percent for the Fleanor Member. Those rock formations are the proposed embedment foundation strata. The staff concludes that these GSI values are reasonable based on applicant's subsurface investigation results and staff's examination of rock cores during its May 2017 site audit. The staff reviewed the RocData input and output parameters for each stratigraphic unit to evaluate the rock mass strength and deformation modulus determination, as summarized in SSAR Table 2.5.4-22 and SSAR Table 2.5.4-23. The applicant developed the deformation modulus using three empirical models available in RocData, and presented its results in SSAR Table 2.5.4-25. The staff reviewed the results and noted that for a GSI of 80 percent the rock mass compressive strength is approximately one-third of the intact compressive strength. However, the staff also noted that in several cases when using the lower range of the GSI, the rock mass compressive strength values are significantly lower than one-third of the intact compressive strength. Although the staff concludes that the applicant used appropriate methods to estimate the rock mass properties of the stratigraphic units at the CRN Site, it is known that the strength of fractured zones, discontinuities, and jointed rock is generally less than that of the individual units of the rock mass and that the empirical methods have a high degree of uncertainty. Therefore, the staff identified the following COL action item:

COL Action Item 2.5-5

> An applicant for a COL or CP referencing this early site permit will need to perform additional testing to determine rock mass properties and to further characterize the rock shear strength along the bedding planes with discontinuities and fracture zones in areas where the safety-related structures will be located.

Based on its review of SSAR Section 2.5.4.2 and the applicant's response to the RAI question discussed above, the staff concludes that the applicant adequately determined the engineering properties of the soil and rock underlying the CRN ESP site following state-of-the-art methodologies for its field and laboratory investigations.

Conclusions Regarding Properties of Subsurface Materials

Based on the review of SSAR Section 2.5.4.2, the staff concludes that the applicant adequately described the subsurface materials and properly determined the engineering properties of the subsurface materials at the CRN Site for the current ESP, and therefore meets the requirements of 10 CFR 100.23. In addition, as described in COL Action Items 2.5-4 and 2.5-5, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.

2.5.4.4.3 Foundation Interfaces

In SSAR Section 2.5.4.3, the applicant described the foundation interface conditions at the CRN Site based on a comprehensive geotechnical exploration and testing program. This program included borehole drilling and sampling, in situ geophysical testing, and observation well installation and testing, as well as laboratory testing.

In addition to the subsurface investigation programs performed for the CRBRP, the CRN Site subsurface investigation included 82 exploratory borings, three test pits, 44 observation wells, and two surface geophysical tests – reflection and refraction tests. The applicant performed downhole geophysical tests in 28 borings, rock pressuremeter tests in two borings, field permeability tests, and groundwater level monitoring in the observation wells.

The applicant chose two specific locations, Location A and Location B, as candidates of power block location that are illustrated in Figure 2.5.4-1. This non-unique power block area selection was based on the dipping strata at the CRN Site that results in the presence of varying stratigraphic units beneath the potential power block area, and the possibility that multiple reactor units will be built at the site.

The NRC staff focused its review of SSAR Section 2.5.4.3 on the relationship between the planned foundations for safety-related structures and the engineering properties of underlying materials. The CRN Site consists of a succession of stratigraphic units with a dip angle of about 33 degrees. Discontinuities, shear-fracture zones, and weathered/fracture zones are all encountered in the stratigraphic units. This special geologic condition will affect the rock engineering properties determination and the stability analyses of foundations and subsurface materials.

The staff reviewed the cross-sections provided in SSAR Figures 2.5.4-2 and 2.5.4-12 in detail, and the results of all subsurface investigations conducted at the CRN Site. The staff finds that the applicant conducted sufficient site investigations and provided adequate descriptions of the subsurface material conditions for the ESP application.

Conclusions Regarding Foundation Interfaces

The staff concludes that the applicant provided an acceptable characterization of the relationship between foundations and underlying materials at the CRN Site, based on the results of geotechnical exploration. In addition, the applicant’s testing methods are consistent with industrial standards and common engineering practices. The staff concludes that the applicant’s evaluation of foundation interfaces is acceptable.

2.5.4.4.4 Geophysical Surveys

The staff focused its review of SSAR Section 2.5.4.4 on the adequacy of the applicant’s geophysical investigations to determine the soil and rock dynamic properties. The applicant provided a summary of geophysical surveys for the CRBRP site and compared these results with the measurements obtained from the CRN Site investigation. These surveys consisted of seismic refraction and reflection, electrical resistivity, spontaneous potential, and a suite of downhole geophysical tests, including Suspension P-S velocity logging, acoustic televiewer, and other downhole logging data.

The staff reviewed the applicant’s use of the latest geophysical and geotechnical testing methods and equipment, as well as the applicant’s results that detail the dynamic properties of the soil and rock underlying the site, in accordance with the requirements in 10 CFR 100.23 and guidance outlined in RG 1.132.

The staff examined the results of the applicant's geophysical surveys and paid special attention to the V_s and V_p profiles. The staff reviewed SSAR Figures 2.5.4-5 and 2.5.4-6, which show the V_s and V_p profiles developed from the downhole geophysical testing and suspension velocity logging for each of the stratigraphic units. The staff also reviewed SSAR Figure 2.5.4-7, which shows the seismic tomography models for the CRN Site. The staff noted that the applicant conducted a series of geophysical surveys using different methods that provide sufficient and reliable data to determine the in situ dynamic properties of soil and rock at the CRN Site.

Conclusions Regarding Geophysical Surveys

Based on its review of SSAR Section 2.5.4.4, the staff concludes that the applicant used acceptable geophysical survey methods that are up-to-date and commonly used in current engineering practices to determine seismic wave velocity for soil and for each of the rock formations at the CRN Site. The staff further concludes that the applicant adequately determined the dynamic properties of soil and rock based on the results of the geophysical surveys conducted at CRN Site and meets the requirements of 10 CFR 100.23.

2.5.4.4.5 Excavation and Backfill

The staff focused its review of SSAR Section 2.5.4.5 on the extent of anticipated excavations for safety-related structures; fills and slopes; excavation methods and stability; backfill sources; quality control; and ITAAC.

Extent of Excavations, Fill and Slopes

The applicant proposed two locations, Location A and Location B, as the center of the planned reactors with a suggested finished plant grade at an elevation of El. 250.2 m (821 ft). The applicant anticipated that the bottom of the basemat of the most deeply embedded safety-related power block structures will not exceed a depth of 42.1m (138 ft) below finished grade and will be founded on rock, and the lateral extent of the excavation will be on the order of 4.6 m (15 ft) beyond the exterior face of the perimeter walls to provide working room for construction and backfilling of the exterior walls.

The applicant stated that because excavation sidewalls are expected to be vertical or near-vertical due to the depth of excavation, stability measures, such as the use of tied-back sheet piles and surface mounted cranes, may be required.

As the applicant specified that design of the excavation will be done for the detailed design and construction work, the staff identified the following COL action item:

COL Action Item 2.5-6

> An applicant for a COL or CP application referencing this early site permit should provide specific details regarding the lateral and vertical extent of the excavation consistent with the selected reactor technology.

Excavation Methods and Stability

The staff reviewed the applicant's description of excavation methods and associated stability issues in SSAR Section 2.5.4.5.2.

The applicant stated that conventional earthmoving equipment can be used for excavation in existing fill/residual soil and weathered rock, and that controlled blasting techniques will be required for rock excavation. During the excavation, near-vertical slopes will be created and stabilization of the slopes will be needed.

As both concrete and granular backfill are required after excavation, the applicant outlined the general requirements for concrete and compacted granular backfill material and included suggested sources for these materials.

The applicant specified items that need to be addressed in the COL application, including: additional subsurface data that may be required to further characterize the underlying stratigraphic bedrock units for the final plant layout because the foundation backfill design will be based on information from geologic mapping of exposed rock surfaces; the specific design of the excavation support system, including rock bolting, will be developed during the detailed design; the slope movement and foundation performance will be monitored with an extensive instrumentation program during and after the excavation; and, a detailed field and laboratory test program will be carried out for the evaluation of backfill sources and their engineering properties.

Based on the review of those construction related items, the staff identified the following COL action items:

COL Action Item 2.5-7

> An applicant for a COL or CP application referencing this early site permit should specify excavation procedures and methods that will not have adverse impacts on the integrity of the foundation subsurface materials. Proper excavation support should be designed, and the stability of excavation slopes should be evaluated. A monitoring plan that includes detailed instrumentation and data collection should be developed to monitor slope movement and heave of subsurface materials due to excavation, changes in pore pressures of soil underneath the foundation, and displacement of the foundation during and after construction.

COL Action Item 2.5-8

> An applicant for a COL or CP application referencing this early site permit should provide detailed design of backfill materials including identification of sources and quantity requirements, backfill material property and placement specifications, applicable industry standards, as well as related ITAAC. The in-place backfill hydraulic characteristics such as permeability and porosity should be consistent with those specified in the SSAR. If differences exist, then its effect on the site conceptual model and site characterization, as described in the SSAR, should be evaluated.

Conclusions Regarding Excavation and Backfill

Based on its review of SSAR Section 2.5.4.5, the staff concludes that the applicant provided sufficient details related to the extent of anticipated excavations for safety-related structures, excavation methods and stability, backfill sources, and quality control, consistent with commonly accepted engineering practices. Therefore, this section of the ESP application is acceptable and meets the relevant requirements of 10 CFR 100.23. In addition, as described in COL Action Items 2.5-6 through 2.5-8, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.

2.5.4.4.6 Groundwater Conditions

In SSAR Section 2.4.12 presents the applicant's full descriptions and results of the groundwater flow models to be used during construction and subsequent plant operations. The staff's evaluation of this model is provided in Section 2.4.12.4 of this report.

The staff reviewed SSAR Section 2.5.4.6 focusing on site groundwater conditions, construction dewatering, and groundwater chemical properties.

The applicant evaluated the groundwater levels at the site based on groundwater observation well data. The applicant anticipates that temporary dewatering of the foundation excavations will be needed during construction, and discussed dewatering methods with associated monitoring requirements. The applicant also discussed the groundwater chemical test results and concluded that the water-soluble sulfate concentration in contact with concrete is low and injurious sulfate attack is not a concern. In addition, the protection requirement specified in the ACI standard 318-14 should be followed.

The staff concludes that the applicant provided sufficient information on groundwater conditions with adequate descriptions of construction dewatering, and groundwater chemical properties for the current ESP. As detailed design of dewatering and foundation protection will be done for the COL application, the staff identified the following COL action items:

COL Action Item 2.5-9

> An applicant for a COL or CP application referencing this early site permit should provide detailed design of dewatering and groundwater control during excavation and construction including a monitoring plan, and provide an evaluation of the impact of dewatering on the stability of foundations.

COL Action Item 2.5-10

> An applicant for a COL or CP application referencing this early site permit should provide detailed design of foundation protection based on chemical characteristics of the groundwater and foundation and fills materials at the site consistent with the applicable industrial standards.

Conclusions Regarding Groundwater Conditions

Based on its review of SSAR Section 2.5.4.6, the NRC staff concludes that the applicant conducted an appropriate preliminary evaluation of groundwater conditions at the CRN Site, and adequately discussed the anticipated dewatering method and requirements during excavation and construction. The applicant also provided chemical test results of groundwater and the possible effect on foundation materials. Because there is no specific reactor design selected, and a detailed dewatering plan will be developed for the COL application, the evaluation of the effect of groundwater conditions and dewatering on the stability of foundation materials, as well as groundwater control throughout the life of the plant, cannot be performed at this time. However, the applicant's discussion on dewatering methods and requirements, and the requirement of foundation and fills to be protected from exposure to groundwater chemicals, are in line with the common engineering practices and industrial standards, and therefore is acceptable. In addition, as described in COL Action Items 2.5-9 and 2.5-10, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.

2.5.4.4.7 Response of Soil and Rock to Dynamic Loading

The staff reviewed SSAR Section 2.5.4.7, focusing on the method and procedure used to develop seismic wave velocity profiles, soil shear modulus and damping degradation properties that are important for site seismic response, and other analyses. The applicant provided detailed information on the site amplification/attenuation analysis, the site seismic response analysis, and the development of the GMRS, in SSAR Section 2.5.2.5. The staff's evaluation of that information is presented in Section 2.5.2.4.5 of this report.

The applicant first developed seismic wave (both shear wave, V_s and compression wave, V_p) velocity profiles for each rock unit based on geophysical surveys conducted for the CRN Site, including seismic refraction and reflection tests, and P-S Suspension logging. The applicant then assembled the seismic wave profiles for Locations A and B in accordance with the stratigraphic conditions (Figures 2.5.4-3 and 2.5.4-4). During the development of the seismic wave velocity profiles, the applicant used geophysical survey data taken within 30.5 m (100 ft) of the locations, and other data, such as SASW data, for depths where no measurement is available. The applicant assigned a shear wave velocity value of 3,353 m/s (11,000 fps) for bedrock when no measurement or other data were available. This assigned shear wave velocity value is within the normal range of similar rocks and reasonable for the bedrock presented at the CRN Site

The applicant took the epistemic uncertainty into consideration when developing the V_s profiles. The applicant developed a best estimate base case V_s profile and upper- and lower-range profiles that use a plus and minus 25 percent variation about the base case, as illustrated in Figures 2.5.4-5 and 2.5.4-6. The variation of 25 percent of the base case V_s profile is commonly used in site seismic response analysis and is acceptable.

The applicant used appropriate EPRI curves to describe the subsurface material damping and shear modulus reduction properties for soil and rock at the site. The applicant also used the RCTS test data to compare with the generic EPRI curves for existing fill soil to ensure that the EPRI curves used in the analyses closely represent the actual site conditions.

The actual locations of the safety-related structures may differ from the proposed locations, and additional site investigations need to be conducted for either a COL or a CP application. Because seismic wave velocity profiles and other dynamic properties of the subsurface materials may need to be updated, the staff identified the following COL action item:

COL Action Item 2.5-11

An applicant for a COL or CP application referencing this early site permit should develop seismic wave velocity profiles for the locations where the safety-related structures will be built, based on sufficient detailed site investigation data with consideration of uncertainties and variability. The appropriate damping and shear modulus reduction properties for soil and rock should be determined for in situ subsurface materials at the CRN Site based on test data and/or justifiable generic curves.

Conclusion Regarding Response of Soil and Rock to Dynamic Loading

Based on its review of SSAR Section 2.5.4.7, the NRC staff concludes that the applicant provided adequate information on the development of the seismic wave velocity, especially the V_s profiles. The applicant also adequately described the modulus reduction and damping properties with proper justification and verification. The staff further concludes that the applicant adequately described the properties of soil and rock responding to seismic loading, and provided proper parameters for site seismic response analysis based on test data and generic EPRI curves that fit the CRN Site conditions. This is in line with the common engineering practices and meets the requirements of 10 CFR 100.23. In addition, as described in COL Action Item 2.5-11, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.

2.5.4.4.8 Liquefaction Potential

The staff reviewed SSAR Section 2.5.4.8, which includes the applicant's description of liquefaction potential at the CRN Site. The staff noted that sound rock is presented at shallow depth (less than 15 m (50 ft)) from ground and the anticipated foundation levels considered by the applicant are about 24.4 m (80 ft) and 42.1 m (138 ft) below the final grade; therefore the foundation will be built on sound rock. In additon, the existing fill residual soils are classified as CH clay. Based on the properties of the soil and rock at the site, the staff concurs with the applicant's conclusion that there is no liquefaction potential at the CRN Site.

Conclusion Regarding Liquefaction Potential

Based on its review of SSAR Section 2.5.4.8, the NRC staff concludes that the applicant performed an adequate evaluation of the liquefaction potential at the CRN Site. The applicant reached a no liquefaction potential conclusion, based on the geologic conditions of the site and the properties of the subsurface material, which meets the requirements of 10 CFR 100.23.

2.5.4.4.9 Earthquake Design Basis

In SSAR Section 2.5.4.9 referred to SSAR Section 2.5.2.5.8 for detailed information on the development of the site-specific GMRS. Section 2.5.2.4.6 of this report provides the staff's evaluation of the site-specific GMRS.

2.5.4.4.10 Static and Dynamic Stability

The staff reviewed SSAR Section 2.5.4.10 focusing on the applicant's evaluation of bearing capacity, settlement, and lateral earth pressures in the two proposed power block areas at the CRN Site.

Bearing Capacity

The staff reviewed the methods and associated assumptions that the applicant used in its evaluation of foundation bearing capacity for the proposed power block areas. The staff noted that the applicant considered each rock formation underlying the power block areas separately by treating each stratigraphic rock formation unit as a single infinite rock layer below the foundations. The applicant then evaluated the bearing capacity for each rock formation. The applicant compared the calculated bearing capacity values and then chose the lowest value as the recommended design bearing capacity for the CRN Site. However, this single rock formation layer assumption does not represent the specific site geologic condition because the actual subsurface of the CRN Site consists of multiple inclined layers of various rock formations with possible weakened interfaces between the formations. In addition, the methods used by the applicant to determine the site bearing capacity are based on a fundamental assumption that the structure is founded on a uniform half-space material, but this assumption may not be applicable for the CRN Site. The applicant also evaluated bearing capacity by using a 2D FEM model that takes the site-specific geologic characteristics into consideration. The applicant's results are summarized in SSAR Section 2.5.4.13. It was not clear to the staff if the applicant included the FEM results as part of its bearing capacity determination presented in SSAR Section 2.5.4.10.1. Therefore, in eRAI-9035 (RAI No. 6), Question 02.05.04-2, the staff asked the applicant (1) to discuss all methods used in its determination of recommended allowable bearing capacity values, and (2) to justify why the bearing capacity calculation methods, based on a uniform half-space subsurface materials assumption, can be used for the CRN Site.

The applicant provided a response to eRAI-9035 (RAI No. 6), Question 02.05.04-02 (1) and (2), in response letter CNL-17-099 dated September 15, 2017 (TVA, 2017 - ADAMS Accession No. ML17261A062). The applicant provided detailed explanations to justify the use of simplified site geologic conditions in its estimate of site bearing capacity. The applicant first provided more details on the four empirical methods used to evaluate bearing capacity with simplified site geologic condition assumptions. The applicant stated that (1) the Wyllie method is suitable to evaluate the bearing capacity of a closely fractured or very weak rock; (2) the Kulhawy and Carter method assumes a strip footing, incorporates the intact rock strength, and accounts for discontinuities in the rock; (3) the US Army Corps of Engineers methods can be used to estimate bearing capacity for four general rock mass conditions (intact, jointed, layered, and highly fractured); and (4) the Bowles method evaluates allowable bearing capacity based on geology, rock type, and rock quality measured by the RQD. The applicant then discussed the FE modeling (presented in SSAR Section 2.5.4.13) that incorporates more realistic site-specific geologic conditions and strategic configurations in bearing capacity evaluation. The applicant stated that the FE model incorporates the inclined rock units beneath foundations and conservatively includes a weakened interface between rock units. The failure mode exhibited by the FE modeling results indicated general shear failure mode of the rock, which is consistent with the assumptions of the empirical approaches. More importantly, the FE modeling results provide good agreement with the results obtained by empirical models. Specifically, the FE model resulted in a value of 7,037 kPa (147 ksf) compared to 7,133 kPa (149 ksf) using the Bowles method at Location A, and the FE model resulted in a value of 5,122 kPa (107 ksf) compared to 5,170 kPa (108 ksf) using the Bowles method at Location B. The applicant concluded that the FE modeling results validated the empirical methods used to evaluate bearing capacity, and confirmed that the assumptions used in the model calculations are appropriate for the CRN Site. The FE modeling results also confirmed that the recommended allowable bearing pressure value of 5,266 kPa (110 ksf) is appropriate. The applicant then concluded that the similarity of the engineering properties of these rock units, in both strength and stiffness, suggests that, for evaluation purposes, the individual rock units may be considered separately to develop a range of results. In addition, the FE model analysis results confirmed that the empirical relationships can be used to estimate bearing capacity for this site.

The staff notes that the uniform half-space subsurface materials assumption used in the empirical methods to estimate the site bearing capacity does not represent the actual site subsurface geologic condition, as the actual site consists of multiple inclined layers of various rock formations with possible weaker interfaces between the formations. In addition, the empirical methods assume a uniform single rock unit condition. Even so, the staff determined that the selected calculation results are in good agreement with that obtained by the FE modeling. Since the FE model represented the actual CRN Site subsurface geologic conditions with inclined rock units beneath foundations, and conservatively includes a weakened interface between rock units, the recommended allowable bearing capacity values based on the results calculated by the selected empirical methods are acceptable because the lowest values with an adequate FS were chosen for the CRN Site. Because the applicant provided a detailed discussion of the empirical methods used to estimate the site bearing capacities, and adequately justified the use of those methods, the staff considers eRAI-9035 (RAI No. 6), Question 02.05.04-2 (1) and (2) resolved.

The locations and elevation of foundations for safety-related structures of a nuclear power plant at the CRN Site will be determined for the COL or CP application and the foundation bearing capacity will need to be reevaluated. Therefore, the staff identified the following COL action item:

COL Action Item 2.5-12

> An applicant for a COL or CP application referencing this early site permit should evaluate the foundation bearing capacity for safety-related structures, based on selected plant structure and foundation designs and actual geologic conditions at the CRN Site under anticipated maximum static and dynamic/seismic loadings.

Settlement and Heave

The staff reviewed the methods and results of the applicant's analyses for settlement and heave of the subsurface materials at the proposed power block locations. Although the applicant used multiple methods to estimate the settlement and heave for the CRN Site, and considered the discontinuity of rock mass in its evaluation, the applicant used the same simplified site geologic condition assumptions as that used in its bearing capacity analysis, which does not represent the actual site condition. Therefore, in eRAI-9035 (RAI No. 6), Question 02.05.04-2 (3) the staff asked the applicant to justify why the settlement and heave calculation methods, based on a uniform half-space subsurface materials assumption, is applicable for the CRN Site.

The applicant provided a response to eRAI-9035 (RAI No. 6), Question 02.05.04-02 (3), in Response Letter CNL-17-099 dated September 15, 2017 (ADAMS Accession No. ML17261A062). The applicant stated that although the inclined layered stratigraphy beneath the foundation is not considered in the method applied, as demonstrated in its bearing capacity evaluation, these assumptions are appropriate because the stratigraphic units beneath the foundations contain similar lithologies, do not exhibit well defined unit boundaries, and exhibit similar strength characteristics. The applicant stated that the rock mass elastic moduli were determined based on in situ V_s measurements and measured in laboratory unconfined compression tests using two different methods. The applicant pointed out that the analysis illustrates the relative similarity of modulus values between stratigraphic units, which is one basis for using a simplified geologic model to evaluate the settlement and heave. In addition, the applicant performed a FE model that considered the inclined layered geology and associated rock mass properties, and conservatively included a weakened interface between rock units, to estimate the foundation settlement. A comparison of settlement results using the simplified geologic model with that using the inclined layered geology as presented in the FE model shows good agreement, thus supporting the use of a simplified geologic model for the CRN Site. The applicant concluded that regardless of the methodology used, computed settlement and heave values are negligible with settlement values ranging from 0.25 to 7.11 mm (0.01 to 0.28 inch), and heave values ranging from 0.5 to 9.1 mm (0.02 to 0.36 inch).

Because the applicant adequately justified the use of a simplified site geologic model and empirical methods to estimate site settlement and heave, and used a more realistic FE model that considered the actual site geologic conditions to confirm the analysis results obtained from the empirical methods, the staff considers Number 6, Question 02.05.04-2 (3) resolved.

The locations and elevation of foundations for safety-related structures of a nuclear power plant at the CRN Site will be determined for the COL or CP application and the settlement and heave of the foundations will need to be reevaluated. Therefore, the staff identified the following COL action item:

COL Action Item 2.5-13

> An applicant for a COL or CP application referencing this early site permit should evaluate the foundation settlement and heave for safety-related structures, based on selected plant structure and foundation designs, and actual geologic conditions at the CRN Site under anticipated excavation depth and maximum static and dynamic/seismic loadings.

Lateral Earth Pressures

The staff reviewed the methodology that the applicant proposed for the evaluation of lateral earth pressure exerted on foundation/structure walls below ground. The staff concluded that the applicant’s proposed method to determine the static lateral earth pressure and associated assumptions, such as no friction between underground structure wall and backfill and an internal friction angle of 20 to 30 degrees for in situ and backfill soil, are reasonable. The staff made this determination because the assumed soil parameters are at the lower range of normal values for in situ soil and for engineering backfills, and the applicant’s proposed method for lateral earth pressure determination is commonly used in engineering practices.

The applicant acknowledged that the COL application needs to be based on a full assessment of lateral earth pressures, including lateral pressures contributed from static soil pressure, hydrostatic pressure, surcharge-induced (equipment and adjacent structures) pressure, andseismic induced pressure. Therefore, the staff identified the following COL action item:

COL Action Item 2.5-14

> An applicant for a COL or CP application referencing this early site permit should

evaluate the maximum lateral earth pressure and its distribution along foundation/structure walls below ground. The total lateral earth pressure should include pressures contributed from static soil pressure, hydrostatic pressure, surcharge-induced (equipment and adjacent structures) pressure and seismic lateral earth pressure at the CRN Site under anticipated maximum static and dynamic/seismic loadings.

Conclusions Regarding Static and Dynamic Stability

Based on its review of SSAR Section 2.5.4.10, and the applicant's responses to related RAIs, the NRC staff concludes that the applicant provided an adequate preliminary assessment of the static stability of the CRN ESP site. In line with common engineering practices and the requirements of 10 CFR 100.23, the applicant performed adequate evaluations of bearing capacity, settlement and heave, and provided adequate information on the evaluation of lateral earth pressure at the CRN Site. In addition, as described in COL Action Items 2.5-12 to 2.5-14, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.

2.5.4.4.11 Design Criteria

The staff reviewed the geotechnical design criteria used by the applicant for the CRN ESP application. The staff concludes that the design criteria, such as the soil liquefaction screening criteria, factors of safety for bearing capacity, acceptable settlement limits, and slope stability requirement, are in line with the general engineering practices and guidelines provided in the relevant NRC guidance.

The applicant stated that the design criteria addressed in the CRN ESP application will be reevaluated in the COL application. In response, the staff identified the following COL action item:

COL Action Item 2.5-15

> An applicant for a COL or CP application referencing this early site permit should identify and reevaluate geotechnical engineering related design criteria to meet applicable industrial standards and NRC regulations.

2.5.4.4.12 Techniques to Improve Subsurface Conditions

The staff reviewed the applicant's discussion of techniques to be used for subsurface condition improvement in areas with safety-related structures. The staff examined the proposed subsurface improvement methods, such as grouting to be used to remedy the voids/cavities located below the foundation level, and the anticipated concrete fill for creating a smooth and level foundation surface. The applicant also suggested implementing a monitoring plan with proper installation of sensors and instruments to monitor slope movement, heave due to the excavation, the vertical movement of the foundation, and changes in pore pressures. The staff concludes that the applicant's proposed subsurface improvement methods and monitoring plan are reasonable because the methods and equipment proposed are commonly used in engineering practices, and are adequate for the purpose of improving the stability of foundations at the CRN Site.

The actual locations of safety-related structures and the necessary subsurface condition improvement method and associated monitoring program will be determined for the COL or CP application. Therefore, the staff identified the following COL action items:

COL Action Item 2.5-16

> An applicant for a COL or CP application referencing this early site permit should improve subsurface conditions in the influence zone of foundations for safety-related structures when karst or other geologic hazard features are discovered. Remediation methods should be determined after evaluating the presence of geologic hazard features based on the results of adequate and more detailed geophysical testing at the site.

2.5.4.4.13 Foundation Assessment Model

The staff focused its review of SSAR Section 2.5.4.13 on the elements used in the applicant's FE model. In addition, the staff focused on the results of the model analysis for assessment of the impact of cavities on foundation stability, as well as foundation bearing capacity and settlement estimates at the CRN Site. To get details on the FE model analysis, the staff also reviewed "Submittal of Supplemental Information Associated with Site Safety Analysis Report Section 2.5 in Support of the Clinch River Nuclear Site Early Site Permit Application," (TVA, 2017 - ADAMS Accession No. ML17186A113) provided by the applicant. The staff examined the model elements, including boundary and interface elements used in 2D FEM modeling, the assumptions related to the size and locations of a hypothetical void within the influence zone of foundation loading, and assumptions regarding the embedment depth of the foundation. The 2D FE model simulations were carried out by using a commercial computer software, PLAXIS 2D. PLAXIS 2D is a commonly used software in geotechnical engineering related design and analysis. Therefore, the staff determined that it is acceptable to use in assessing the impact of cavities on foundation stability. The applicant modeled the subsurface characteristics with multiple inclined rock formations with weaker contact interfaces. The applicant obtained the engineering properties of the subsurface materials using site investigation data, which closely represents the actual geologic conditions of the CRN Site. The applicant's assumption of a maximum void with a 4.6 m (15 ft) diameter and infinite length is consistent with the maximum vertical void discovered during site investigation, and the length of the void is conservatively assumed. Additionally, the applicant's assumption of the foundation embedment depth from 12.2 m to 42.7 m (40 ft to 140 ft) covers the embedment depth for known SMR designs. Also, the applicant considered the possible locations where voids may have the most impact on the foundation stability in its assumption of the void depth of 1.5 m to 9.1 m (5 ft to 30 ft) below the foundation and void locations at the edge and center of the nuclear island and at the interface of different rock formations. Therefore, the staff concludes that the 2D FE model that the applicant developed to evaluate the effect of underground void on foundation stability is adequate and acceptable. The staff also concludes that the assumptions used in the model are reasonable.

Based on its review of the applicant's 2D FE model and simulation results, the staff finds that the applicant realistically modeled the specific geologic conditions at the CRN Site; the size and location of possible voids at the site are conservatively assumed; and the results show that the proposed Locations A and B are generally suitable for an SMR foundation.

The applicant used the same 2D FE model to estimate the site bearing capacity and settlement in order to confirm the validity of the simplified model used for bearing capacity and settlement assessments presented in SSAR Subsection 2.5.4.10. The applicant's analysis results showed that the estimated allowable bearing capacity and settlement values are in good agreement with that determined by simplified methods. For example, for Location A, the PLAXIS bearing capacity is 7,037 kPa (147 ksf) compared with 7,133 kPa (149 ksf) determined by simplified methods; and for Location B, the PLAXIS bearing capacity is 5,122 kPa (107 ksf) compared with 5,170 kPa (108 ksf) determined by the simplified methods.

The applicant specified that the foundation performance needs to be reevaluated in the COL or CP application, based on selection of a final technology and final foundation locations that will be determined based on additional detailed site-specific geologic conditions including stratigraphy, subsurface layering orientation, and specific fracture or bedding plane discontinuity zonation. In response, the staff identified COL Action Item 2.5-2 that is described in Section 2.5.4.4.1 of this report.

Conclusions Regarding Foundation Assessment Model

Based on its review of SSAR Section 2.5.4.13 and the applicant's supplemental report, the NRC staff concludes that the applicant used a realistic subsurface model based on the geologic characteristics of the site, and used a conservative approach for estimating a hypothetical void with respect to its size and locations. The staff determined that the applicant adequately evaluated the impact of voids on the foundation stability for the CRN Site. The analysis results showed that the CRN Site is generally suitable for a SMR nuclear power plant. The staff further concludes that the adequate foundation stability assessment meets the requirements of 10 CFR 100.23, and is therefore acceptable. In addition, as described in COL Action Items 2.5-15 and 2.5-16, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.

2.5.4.5 Permit Conditions

In SSAR Section 2.5.4.5.4.3, the applicant acknowledged the need to perform geologic mapping for documenting the presence or absence of faults and shear zones in plant foundation materials before placement of concrete backfill and foundation concrete. Therefore, in Section 2.5.3.5 of this report, the staff identified Permit Condition 1 related to detailed geologic mapping of safety-related excavations at the CRN Site as the responsibility of the COL or CP applicant.

For the suitability evaluation of a proposed site, requirements in 10 CFR 100.23, specifically 10 CFR 100.23(c), provides that the engineering characteristics of a site and its environs must be investigated in sufficient scope and detail to permit an adequate evaluation of the proposed site. 10 CFR 100.23(d) discusses several siting factors and potential causes of failures that must be evaluated, including rock stability, the physical properties of the materials underlying the site, and ground disruption, in addition to several other geologic and seismic factors. The applicant identified discontinuities, shear fractures zones, and weathered fracture zones, which typically exist in the uppermost 30.5 m (100 ft) at the CRN Site and are not suitable for safety related structures to be built on; and the rock mass characterization presented in the ESP application mainly applies to bedrock stratigraphic units below 24.4 m (80 ft) (El. 225.9 m (741 ft) NAVD88). Therefore, the staff identified Permit Condition 2 as follows:

Permit Condition 2

> An applicant for a combined license (COL) or a construction permit (CP) that references this early site permit shall remove the material above El. 225.9 m (741 ft) NAVD 88 in areas where safety-related structures will be located to minimize the adverse effects of discontinuities, weathered and shear-fracture zones, and karst features on the stability of subsurface materials and foundations. The applicant shall also perform additional geotechnical investigations, in accordance with RG 1.132, at the excavation level to identify any potential geologic features that may adversely impact the stability of subsurface materials and foundations.

In the event that adverse geologic features are identified through implementation of Permit Conditions 1 and 2, the applicant should excavate or improve the subsurface materials to ensure the stability of safety-related structures in accordance with COL Action Item 2.5-3.

2.5.4.6 Conclusions

Based on its review of SSAR Section 2.5.4 and pertinent supplemental information, and the applicant's responses to RAIs related to this section, the NRC staff concludes that the applicant provided adequate information describing geologic and engineering characteristics of the subsurface materials at the CRN Site based on data collected from site investigations. The applicant conducted adequate field and laboratory tests by using state-of-the-art methods, in accordance with applicable industrial standards and the guidance of RG 1.132, RG 1.138, and RG 1.198. The staff also concludes that the applicant adequately evaluated the site suitability regarding the stability of subsurface materials and foundations with respect to the engineering properties of subsurface materials at the proposed site, the assessment of liquefaction potential, bearing capacity, settlement, and lateral earth pressure, as well as the development of a shear wave velocity profiles for the proposed power block locations. The staff further concludes that the applicant adequately described requirements for a COL applicant referencing this ESP for topics where detailed stability evaluations could not be performed for the ESP application because no nuclear power plant design has been specified, and the actual location of the safety-related structures could not be determined. Based on the above, the staff concludes that the applicant meets the applicable requirements of 10 CFR Part 50, Appendices B and S, 10 CFR 52.17(a)(1)(vi) and 10 CFR 100.23.

For evaluation of suitability of a proposed site, 10 CFR 100.23(c) requires that the geological, seismological, and engineering characteristics of a site and its environs must be investigated in sufficient scope and detail to permit an adequate evaluation of the proposed site, to provide sufficient information to support evaluations performed to arrive at estimates of the Safe Shutdown Earthquake Ground Motion, and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site. Since more detailed site characterization for further siting evaluation, and specific engineering analyses for foundation stability evaluations will be needed for a chosen nuclear power plant design in the COL or CP application, the staff identified issues specified in COL Action Items 2.5-1 through 2.5-16 in this SER that COL or CP applicant referencing this ESP must address in its application.

2.5.5 Stability of Slopes

2.5.5.1 Introduction

In SSAR Section 2.5.5, "Stability of Slopes," the applicant addresses the stability of both natural and manmade (cuts, fill, embankments, dams, etc.) earth slopes whose failure could adversely affect safety-related structures. The staff evaluated this section based on the data provided by the applicant in the SSAR. In SSAR Section 2.5.5, the applicant indicated that given the existing topography, the natural topography and the planned finished grade elevation of 250.2m (821 ft, NAVD88), a flat table-top site with no safety-related slope is anticipated. The applicant stated that the site grading plan and the stability of any safety-related slopes, including dams and dikes are going to be evaluated as part of the COL application.

2.5.5.2 Summary of Application

In SSAR Section 2.5.5 the applicant discusses stability of earth slopes whose failure could affect safety-related structures. The applicant deferred the specifics for slope stability design to the COL or CP application, which will include a selected reactor technology.

2.5.5.2.1 Slope Characteristics

The applicant stated that power block configuration and site grading has not been established, hence the characteristics of any permanent slope will be established in the COL application. The applicant stated that temporary excavation will be made during the construction process and will include vertical faces and sloped ramp for access into the excavation. The applicant indicated that no slopes will remain after construction.

2.5.5.2.2 Design Criteria and Analysis

The applicant stated that if permanent slopes are identified in the COL application, they will be analyzed against potential slope failure at that time.

2.5.5.2.3 Results of the Investigation

The applicant refers to SSAR Section 2.5.4 for details of subsurface investigation. The applicant stated that this data will be used in the design of any permanent safety-related slopes.

2.5.5.2.4 Properties of Borrow Material

The applicant refers to SSAR Subsection 2.5.4.5 for details related to backfill and borrow material for safety-related backfill. The applicant stated that if any permanent safety-related slope is identified once site grading has been established, properties of borrow materials will be determined.

2.5.5.3 Regulatory Basis

The applicable regulatory requirements for the stability of slopes are as follows:

- 10 CFR 52.17(a)(1)(vi), as it relates to the requirement for an ESP applicant to prepare an SSAR that contains information on geologic and seismic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, and with sufficient margin for the limited accuracy, quantity and period of time in which the historical data have been accumulated.

- 10 CFR Part 50, Appendix S as it relates to the design of nuclear power plant structures, systems, and components important to safety to withstand the effects of earthquakes or surface deformation.

- 10 CFR 100.23 as it relates to the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and design of nuclear power plants.

The related acceptance criteria from NUREG–0800, Section 2.5 are summarized as follows:

- Slope Characteristics: To meet the requirements of 10 CFR Part 50, 10 CFR Part 52, and 10 CFR Part 100, the discussion of slope characteristics is acceptable if the discussion includes: (1) Cross-sections and profiles of the slope in sufficient quantity and detail to represent the slope and foundation conditions; (2) a summary and description of static and dynamic properties of the soil and rock comprised by Seismic Category I embankment dams and their foundations, natural and cut slopes, and all soil or rock slopes whose stability would directly or indirectly affect safety-related and Seismic Category I facilities; and (3) a summary and description of groundwater, seepage, and high and low groundwater conditions.

- Design Criteria and Analyses: To meet the requirements of 10 CFR Part 50, 10 CFR Part 52, and 10 CFR Part 100, the discussion of design criteria and analyses is acceptable if the criteria for the stability and design of all Seismic Category I slopes are described and valid static and dynamic analyses have been presented to demonstrate that there is an adequate margin of safety.

- Boring Logs: To meet the requirements of 10 CFR Part 50, 10 CFR Part 52, and 10 CFR Part 100, the applicant should describe the borings and soil testing carried out for slope stability studies and dam and dike analyses.

- Compacted Fill: To meet the requirements of 10 CFR Part 50, and 10 CFR Part 52, the applicant should describe the excavation, backfill, and borrow material planned for any dams, dikes, and embankment slopes.

In addition, the geologic characteristics should be consistent with appropriate sections from RG 1.27, RG 1.28, RG 1.132, RG 1.138, and RG 1.198.

2.5.5.4 Technical Evaluation

The staff reviewed SSAR Section 2.5.5, which provides the applicant's general description of site-specific information related to slope stability, and concludes that the information provided by the applicant is adequate for this ESP application. The applicant deferred the slope stability analysis for the COL application because currently there is no safety-related slope at the CRN Site. As such, the staff identified the following COL action item to address the need for slope stability analyses:

COL Action Item 2.5-17

> An applicant for a COL that references this early site permit should perform a slope stability analysis of any safety-related slopes, including dams and dikes, consistent with the selected reactor technology.

2.5.5.5 Conclusions

In SSAR Section 2.5.5, the applicant stated that, for the COL application, it would evaluate the site grading plan and the stability of any safety-related slopes. Since there is no existing safety-related slopes currently at the CRN Site and the applicant provided necessary information on site topography and geologic conditions, the staff concludes that the SSAR Section 2.5.5 is adequate and acceptable because it meets applicable requirements of 10 CFR Part 50, Appendix S, 10 CFR 52.17(a)(1)(vi) and 10 CFR 100.23. In addition, as described in COL Action Item 2.5-17, the staff identified issues that shall be addressed by a COL or CP applicant referencing this ESP.