2.5 Geology, Seismology, and Geotechnical Engineering

Geology, seismology, and geotechnical engineering information are specific to the site and region and will be addressed by applicants on a site-specific basis. A range of generic site conditions which encompasses a number of potential reactor sites throughout the United States has been selected for evaluating the U.S. EPR.

2.5.1 Basic Geologic and Seismic Information

A combined license (COL) applicant that references the U.S. EPR design certification will use site-specific information to investigate and provide data concerning geological, seismic, geophysical, and geotechnical information.

2.5.1.1 Regional Geology

Regional geology is site specific and will be addressed by the COL applicant.

2.5.1.2 Site Geology

Site-specific geology information will be addressed by the COL applicant.

2.5.2 Vibratory Ground Motion

A COL applicant that references the U.S. EPR design certification will review and investigate site-specific details of seismic, geophysical, geological, and geotechnical information to determine the safe shutdown earthquake (SSE) ground motion for the site and compare site-specific ground motion to the Certified Seismic Design Response Spectra (CSDRS) for the U.S. EPR.

The seismic design basis for the U.S. EPR is presented in Section 3.7.1.1.1. As noted therein, the U.S. EPR is designed for 0.3 g peak ground acceleration (PGA) design ground motion which is defined as a hypothetical free-field outcrop motion at approximately 41.33 ft below grade at the bottom elevation of the foundation basemat for the Nuclear Island (NI) Common Basemat Structures (GDC 2). The certified seismic design response spectra (CSDRS) for the U.S. EPR are shown in Figure 3.7.1-1—Design Response Spectra for EUR Control Motions (Hard, Medium, and Soft Soils). The CSDRS are the same in both horizontal directions and in the vertical direction.

Section 3.7.1.3 describes a range of 10 generic soil profiles and associated dynamic soil properties selected for the design of the U.S. EPR. Table 3.7.1-6—Generic Soil Profiles for the U.S. EPR Standard Plant, shows the soil layering, the assumed strain-dependent properties, and the CSDRS design control motion associated with the profile. The variation in shear wave velocity in each of the assumed profiles is illustrated in Figure 3.7.1-31 and Figure 3.7.1-32—U.S. EPR Standard Plant Soil Profiles. The soil properties associated with the various shear wave velocities assumed in the 10 generic soil profiles are discussed further in Section 3.7.2.4.1 and summarized in Table 3.7.2-9. Section 3.7.1.3 and Section 3.7.2.4.1 discuss that, for soil-structure interaction (SSI) analysis for the U.S. EPR design certification, the assumed generic shear wave velocities in each profile are taken to be strain-compatible values during seismic events.

Refer to Section 3.7.1 and Section 3.7.2 for additional description of soil-structure interaction analyses performed for the U.S. EPR. Liquefaction of soils and stability of slopes is addressed in Section 2.5.4.8 and Section 2.5.5, respectively.

2.5.2.1 Seismicity

Seismicity is site specific and will be addressed by the COL applicant.

2.5.2.2 Geologic and Tectonic Characteristics of the Site and Region

Geologic and tectonic characteristics are site specific and will be addressed by the COL applicant.

The guidance of RG 1.208 and RG 1.165 will be met, as appropriate, in performing the required studies to determine the SSE using probabilistic seismic hazard analyses.

2.5.2.3 Correlation of Earthquake Activity with Seismic Sources

Correlation of earthquake activity with seismic sources is site specific and will be addressed by the COL applicant, consistent with the guidance of RG 1.208 and RG 1.165, as appropriate.

2.5.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquake

The probabilistic seismic hazard analysis is site specific and will be addressed by the COL applicant, consistent with the guidance of NUREG/CR-6372 (Reference 1), RG 1.165, and RG 1.208, as appropriate.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

Seismic wave transmission characteristics are site specific and will be addressed by the COL applicant.

2.5.2.6 Ground Motion Response Spectrum

A COL applicant that references the U.S. EPR design certification will verify that the site-specific seismic parameters are enveloped by the CSDRS (anchored at 0.3 g PGA) and the 10 generic soil profiles discussed in Section 2.5.2 and Section 3.7.1 and summarized in Table 3.7.1-6. The applicant develops site-specific ground motion response spectra (GMRS) and foundation input response spectra (FIRS). The applicant will also describe site-specific soil conditions and evaluate the acceptability of the U.S. EPR standard design described in Section 3.7.1 for the particular site. In making this comparison, the applicant will refer to Sections 3.7.1 and 3.7.2 for a description of the soil-structure interaction analyses performed for the U.S. EPR in addressing the following evaluation guidelines.

1. The applicant will confirm that the peak ground acceleration for the GMRS is less than 0.3g.

2. The applicant will confirm that the low-strain, best-estimate, value of shear wave velocity at the bottom of the foundation basemat of the NI Common Basemat

Structures is 1000 fps, or greater. This comparison will confirm that the NI Common Basemat Structures are founded on competent material.

3. The applicant will demonstrate that the FIRS are enveloped by the CSDRS for the U.S. EPR using the guidance provided in Section 3.7.1.1.1.

4. The applicant will demonstrate that the site-specific profile is laterally uniform by confirming that individual layers with the profile have an angle of dip no greater than 20 degrees.

5. The applicant will demonstrate that the idealized site soil profile is similar to or bounded by the 10 generic soil profiles used for the U.S. EPR. The 10 generic profiles include a range of uniform and layered site conditions. The applicant also considers the assumptions used in the SSI analyses, as described in Section 3.7.1 and Section 3.7.2.

6. If the conditions of steps one through five are met, the characteristics of the site fall within the site parameters for the U.S. EPR and the site is acceptable.

7. If the conditions of steps one through five are not met, the applicant will demonstrate by other appropriate means that the U.S. EPR is acceptable at the proposed site. The applicant may perform intermediate-level additional studies to demonstrate that the particular site is bounded by the design of the U.S. EPR. An example of such studies is to show that the site-specific motion at top-of-basemat level, with consideration of the range of structural frequencies involved, is bounded by the U.S. EPR design.

8. If the evaluations of step 7 are not sufficient, the applicant will perform detailed site-specific SSI analyses for the particular site. This site-specific evaluation will include dynamic seismic analyses and development of in-structure response spectra (ISRS) for comparison with ISRS for the U.S. EPR. These analyses will be performed in accordance with the methodologies described in Section 3.7.1 and Section 3.7.2. Results from this comparison will be acceptable if the amplitude of the site-specific ISRS do not exceed the ISRS for the U.S. EPR by greater than 10 percent on a location-by-location basis. Comparisons will be made at the following key locations, defined in Section 3.7.2:

    A. Reactor Building Internal Structures (RBIS) - Reactor Vessel Support at elevation +16 ft, 10-3/4 in (Figures 3.7.2-74, 75, and 76) and steam generator supports at elevation +63 ft, 11-3/4 in (Figures 3.7.2-77, 78, and 79).

    B. Safeguards Building (SB) 1 – elevation +26 ft, 7 in (Figures 3.7.2-80, 81, and 82) and +68 ft, 10-3/4 in (Figures 3.7.2-83, 84, and 85).

    C. SBs 2/3 – elevation +26 ft, 7 in (Figures 3.7.2-86, 87, and 88) and +50 ft, 6-1/4 in (Figures 3.7.2-89, 90, and 91).

    D. SB 4 – elevation +68 ft, 10-3/4 in (Figure 3.7.2-92, 93, and 94).

E. Reactor Containment Building (RCB) – Polar crane support at elevation +123 ft, 4-1/4 in (Figures 3.7.2-95, 96, and 97) and top-of-dome at elevation +190 ft, 3-1/2 in (Figures 3.7.2-98, 99, and 100).

F. Fuel Building (FB) elevation + 12 ft, 1-2/3 in.

G. Emergency Power Generator Building (EPGB) - basemat elevation. +0 ft, 0 in at Node 1172 (Figures 3.7.2-101, 102, and 103) and +51 ft, 6 in.

H. Essential Service Water Building (ESWB) - Node 10385 on elevation +14 ft, 0 in (Figures 3.7.2-107, 108, and 109) and Node 12733 on elevation +63 ft, 0 in (Figures 3.7.2-104, 105, and 106).

9. Exceedances in excess of the limits discussed in step 8 will require additional evaluation to determine if safety-related structures, systems, and components of the U.S. EPR at the location(s) in question will be affected.

As a result of the reconciliation process described above, the applicant may redesign selected features of the U.S. EPR, as required. Redesigned features will be identified as exceptions to the FSAR and addressed by the COL applicant.

2.5.3 Surface Faulting

No surface faulting is considered to be present under foundations for Seismic Category I structures in the U.S. EPR (GDC 2).

A COL applicant that references the U.S. EPR design certification will investigate site-specific surface and subsurface geologic, seismic, geophysical, and geotechnical aspects within 25 miles around the site and evaluate any impact to the design. The COL applicant will demonstrate that no capable faults exist at the site in accordance with the requirements of 10 CFR 100.23 and of 10 CFR 50, Appendix S. If non-capable surface faulting is present under foundations for safety-related structures, the COL applicant will demonstrate that the faults have no significant impact on the structural integrity of safety-related structures, systems, or components.

2.5.4 Stability of Subsurface Materials and Foundations

The stability of subsurface materials under the and foundations for Seismic Category I structures is demonstrated in Section 3.8.5 for the U.S. EPR 10 generic soil profiles described in Section 3.7.1 and Section 3.7.2. As described in Section 3.8.5, lateral soil pressure loads under saturated conditions are considered for the design of below-grade walls. Soil loads are based on the parameters described in Section 2.5.4.2.

A COL applicant that references the U.S. EPR design certification will present site-specific information about the properties and stability of soils and rocks that may affect the nuclear power plant facilities under both static and dynamic conditions, including the vibratory ground motions associated with the CSDRS and the site-specific SSE.

2.5.4.1 Geologic Features

Geologic features are site specific and will be addressed by the COL applicant.

2.5.4.2 Properties of Subsurface Materials

The following soil properties are used for design of U.S. EPR Seismic Category I structures.

- Soil density:
  - Saturated soil = 134 lb/ft$^{3.}$
  - Moist soil = 128 lb/ft$^{3.}$
  - Dry soil = 110 lb/ft$^{3.}$
- Angle of internal friction = 35 degrees.
- Coefficient of friction acting on foundation basemats and near surface foundations for Seismic Category I structures = 0.7.

For a cohesionless soil site, the soil below and adjacent to the safety-related foundation basemat will have a friction angle in excess of 35 degrees. For a cohesive soil site, the soil will have an undrained strength equivalent to or exceeding a drained strength of 35 degrees (yielding a friction coefficient greater than 0.7).

Section 2.5.4.5 discusses the use of mud mats under the foundation basemats to facilitate construction. When used, the governing friction value at the interface zone is determined by a thin soil layer (soil-on-soil) under the mud mat. As indicated above, the underlying soil (expected to be compacted backfill) will have a friction angle greater than 35 degrees. Typical values of friction coefficient between concrete and dry soil and rock are in the range of approximately 0.7. Due to the interlock of concrete with soil as the concrete is placed, the friction between the mud mat and underlying soil media is generally higher than the friction resistance of soil-on-soil so that continuity of load transfer across the interface is maintained.

Earthquake induced soil pressures for the design of the U.S. EPR are developed in accordance with Section 3.5.3 of ASCE 4-98 (Reference 2). Maximum ground water and maximum flood elevations used for determining lateral soil loads for the U.S. EPR are as specified in Table 2-1.

A COL applicant that references the U.S. EPR design certification will reconcile the site-specific soil properties with those used for design of U.S. EPR Seismic Category I structures and foundations described in Section 3.8.

2.5.4.3 Foundation Interfaces

Foundation interfaces with underlying materials are site specific and will be addressed by the COL applicant. The COL applicant will confirm that the site soils have (1) sliding coefficient of fiction equal to at least 0.7, (2) adequate shear strength to provide adequate static and dynamic bearing capacity, (3) adequate elastic and consolidation properties to satisfy the limits on settlement described in Section 2.5.4.10.2, and (4) adequate dynamic properties (i.e., shear wave velocity and strain-dependent modulus-reduction and hysteretic damping properties) to support the Seismic Category I structures of the U.S. EPR under earthquake loading.

2.5.4.4 Geophysical Surveys

Geophysical surveys are site specific and will be addressed by the COL applicant.

2.5.4.5 Excavations and Backfill

Excavations and backfill are site specific and will be addressed by the COL applicant. The use of waterproofing membranes is site-specific as described in Section 3.8.5.6.1. Mud mats may be provided under foundations for ease of construction. Mud mats may be designed as structural plain concrete elements on a site-specific basis in accordance with ACI 318 (Reference 3). Embedment of waterproofing membranes within mud mats is described in Section 3.8.5.6.1.

2.5.4.6 Ground Water Conditions

Ground water conditions are described in Section 2.4 and provided in Table 2-1 for the U.S. EPR. Ground water conditions are considered in the structural design of the U.S. EPR, as described in Section 3.8. However, groundwater conditions are not explicitly considered in the SSI analyses described in Sections 3.7.1 and Section 3.7.2.

The COL applicant will address site-specific ground water conditions.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

Section 2.5.2 notes that the design of the U.S. EPR is based on the assumption that the shear wave velocities assumed for the 10 generic soil profiles described in Section 3.7.1.3 are strain-compatible properties. For SSI analysis for the U.S. EPR, assumed relationships to depict the strain-dependent modulus-reduction and hysteretic damping properties are not explicitly considered. The COL applicant will address site-specific response of soil and rock to dynamic loading, including the determination of strain-dependent modulus-reduction and hysteretic damping properties.

2.5.4.8 Liquefaction Potential

The design of the U.S. EPR assumes that the plant is not founded on liquefiable materials (GDC 2).

The COL applicant will address site-specific liquefaction potential. As stated in Section 2.5.2, the evaluation of liquefaction is performed for the seismic level of the site-specific SSE.

2.5.4.9 Earthquake Site Characteristics

Section 3.7.1 describes the seismic design basis for the U.S. EPR. Section 2.5.2 presents a brief summary of the seismic design basis.

Site-specific earthquake site characteristics will be described by the COL applicant.

2.5.4.10 Static Stability

Static stability pertaining to bearing capacity and settlement for the U.S. EPR is described in the following section. Additional information is provided in Section 3.8.5 for the foundations of Seismic Category I structures.

2.5.4.10.1 Bearing Capacity

The maximum bearing pressure under static loading conditions for the foundation basemat beneath the NI Common Basemat Structures is 22,000 lb/ft^2, which includes the dead weight of the structure and components and 25 percent of the live load. The maximum bearing pressure under safe shutdown earthquake loads combined with other loads, as described in Section 3.8.5, is 25,000lb/ft^2. Refer to Appendix 3E for details of these bearing pressures under the basemat (GDC 2).

A COL applicant that references the U.S. EPR design certification will verify that site-specific foundation soils beneath the foundation basemats of Seismic Category I structures have the capacity to support the bearing pressure with a factor of safety of 3.0 under static conditions.

2.5.4.10.2 Settlement

Safety-related structures, systems and components are housed primarily in structures supported by the foundation basemat for the NI Common Basemat Structures and independent foundation basemats for the EPGBs and the ESWBs. The design of the Seismic Category I foundations for the U.S. EPR is based on a maximum differential settlement of ½ inch per 50 ft in any direction across the basemat. Settlements within this limit will not adversely affect the function of safety-related structures, systems, or components based on the design basis for relative displacements between SSCs (GDC 2).

Total settlement is dependent on site specific conditions, construction sequence, loading conditions, and excavation and dewatering plans. Up to three inches of settlement might occur following first placement of concrete. At settlement values on the order of three inches, no shear failure of the foundations, general or local, is expected. It is expected that all elastic settlement and most of the consolidation settlement will occur by the time of completion of construction. There are limited interfaces between systems located on different basemats. The effects of settlement and differential settlement are considered where these interfaces occur. As described in Section 3.8.4.1.8 and Section 3.8.4.1.9, the design of safety-related buried conduits and piping is site-specific. These features will be designed for site-specific values of settlement and differential settlement expected at the interface with the foundation basemat after connections are made. Alternatively, site-specific structural features such as tunnels may be used to limit the imposition of differential settlement.

A COL applicant that references the U.S. EPR design certification will verify that the differential settlement value of ½ inch per 50 ft in any direction across the foundation basemat of a Seismic Category I structure is not exceeded. Settlement values larger than this may be demonstrated acceptable by performing additional site-specific evaluations.

Section 3.8.5.7 addresses settlement monitoring.

2.5.4.10.3 Uniformity and Variability of Foundation Support Media

The U.S.EPR design considers a broad range of subsurface conditions, and the effects of these various conditions were evaluated by an extensive series of SSI analyses which addressed subsurface stratigraphy, depth-to-bedrock, shear wave velocity, and its variation with depth. While the U.S. EPR design is intended to cover a broad range of soil conditions, it is recognized that it is impractical to address all possible subsurface variations. For this reason site specific subsurface conditions will be evaluated for applicability to the U.S. EPR.

The design of the U.S. EPR is based on analyses that assume the underlying layers of soil and rock are horizontal with uniform properties. Furthermore, the U.S. EPR is designed for application at a site where the foundation conditions do not have extreme variation within the foundation footprints. However, the design does have margin that allows for adaptation to many sites that might be classified as non-uniform or having highly variable properties.

A COL applicant that references the U.S. EPR design certification will investigate and determine the uniformity of the underlying layers of site specific soil conditions beneath the foundation basemats. The classification of uniformity or non-uniformity will be established by a geotechnical engineer.

Soil structure interaction analysis, settlement analysis, and bearing capacity analysis for the U.S. EPR assume that the soil layers are horizontal and effects of non-horizontal layering are ignored. However, the layers of soil and rock beneath a specific site may dip with respect to the horizontal. If the dip is less than or equal to 20 degrees, the layer is defined as horizontal and analyses using horizontal layers are applicable, as described in NUREG/CR-0693 (Reference 4).

Guidance for performing a site-specific evaluation of uniformity for soil profiles under the Seismic Category I structures is provided below. Alternate site-specific methodologies may be used with appropriate technical justification.

Uniformity within the layer may be checked by determining from the boring logs a series of “best-estimate” planes beneath the foundation footprint that define the top (and bottom) of each layer. Depending on specific site conditions, the planes can be based on stratigraphy, lithology, unconformities, intrusives, weathering, other geologic/geotechnical properties or characteristics and/or combinations of the above. Uniformity and best estimate shear wave velocity within the layer will be established for all layers to a minimum depth of approximately 1.5 times an equivalent radius or no more than 1.0 times the maximum foundation basemat dimension. Typically this will be no less than 200 feet below the bottom of the foundation basemat. If the site can be classified as laterally uniform, it is satisfactory for the U.S. EPR based on analyses and evaluations performed to support design certification, provided that additional site-specific analyses are not required to consider differences in analytical modeling assumptions between the U.S. EPR design and those appropriate to the specific site.

If the site is found to have a dip angle greater than 20 degrees, or the site is found to have non-uniform soil conditions within a profile, site-specific analysis will be performed. This site specific analysis may involve soil structure interaction analysis and/or an analysis that demonstrates that the foundation basemat stresses resulting from the variation of subgrade modulus or shear wave velocity across the footprint are within the design margin for the U.S. EPR foundation basemats. In addition, these considerations may be assessed with the information developed in accordance with RG 1.132 and RG 1.138 to determine if additional site investigation measures are necessary or if site improvement measures should be undertaken.

2.5.4.10.4 Site Investigation for Uniform Sites

For sites that are expected to be uniform, RG 1.132, Appendix D, provides guidance on the spacing and depth of borings of the geotechnical investigation for Seismic Category I structures. Specific language in the Regulatory Guide indicates a spacing of 100 feet supplemented with borings on the periphery and at the corners for favorable, uniform geologic conditions.

For foundation engineering purposes, a series of primary borings should be drilled on a grid pattern that encompasses the NI Common Basemat Structures foundation footprint and an area 40 feet beyond the boundaries of the foundation basemat footprint, plus the area that encompasses the other near surface-founded Seismic Category I structures for the U.S. EPR.

The 40-foot extension for the grid of borings is established from a Boussinesq analysis of the zone of influence of the foundation basemat which shows that the net change in the effective vertical overburden stress is less than 7 percent at a distance of 40 feet from the edge of the foundation basemat. The grid need not be of equal spacing in the two orthogonal directions, but it should be oriented in accordance with the true dip and strike of the rock. If geologic conditions are such that true dip and strike are not obvious, or if the dip is practically flat, then the orientation of the grid can be consistent with the major orthogonal lines of the NI Common Basemat Structures.

The depth of borings should be determined on the basis of the geologic conditions. Borings should be extended to a depth sufficient to define the site geology and to sample materials that may swell during excavation, may consolidate subsequent to construction, may be unstable under earthquake loading, or whose physical properties would affect foundation behavior or stability. At least one-fourth of the primary borings should penetrate sound rock or, for a deep soil site, to a maximum depth of 250 feet below the foundation basemat. At this depth of 250 feet, the change in the vertical stress during or after construction for the combined foundation loading is less than 10 percent of the in-situ effective overburden stress. Other primary borings may terminate at a depth of 160 feet below the foundation (i.e., equal to the equivalent radius of the structure). It is recommended that the shear wave velocity should be measured to a depth of 350 ft to 500 ft beneath the foundation basemat of the NI Common Basemat Structures. Thus, a limited number of borings should penetrate significantly deeper than the 250 ft criterion cited above.

2.5.4.10.5 Site Investigation for Non-uniform Sites

At sites that are judged to be non-uniform, potentially non-uniform, highly variable or potentially highly variable based on not meeting the criteria stated in Section 2.5.4.10.3, the investigation effort may have to be extended to determine if the site is acceptable for the U.S. EPR.

The U.S. EPR foundation/structural system for the NI Common Basemat Structures has significant margin. Therefore, it is expected that all but the most variable of sites will meet the criteria stated in Section 2.5.4.10.3. As stated in RG 1.132, where variable conditions are found, the spacing of boreholes should be closer to adequately define the media properties and their variability. Where cavities or other discontinuities of engineering significance may occur, the normal exploratory work should be supplemented by secondary borings or soundings at a spacing close enough to detect such features.

The depth of the secondary borings is 160 feet below the foundation basemat of the NI Common Basemat Structures. At this depth, the maximum change in vertical stress during or after construction is about 11 percent of the in-situ effective overburden stress. The depth of borings should be extended beyond 160 feet if the geologic investigation indicates the possible presence of karst conditions, under-consolidated clays, loose sands, intrusive dikes, or other forms of geologic impacts at depth greater than 160 feet.

2.5.4.11 Design Criteria

Section 3.8.5 provides design criteria and design methods used in analysis and design of foundations, including a description of computer programs used in the analyses and a description of soil loads on embedded walls.

2.5.4.12 Techniques to Improve Subsurface Conditions

Techniques used for improving subsurface conditions are site specific and will be addressed by the COL applicant.

2.5.5 Stability of Slopes

No slope failure potential is considered in the design of safety-related SSCs in the U.S. EPR.

A COL applicant that references the U.S. EPR design certification will evaluate site-specific information concerning the stability of earth and rock slopes, both natural and manmade (e.g., cuts, fill, embankments, dams, etc.), of which failure could adversely affect the safety of the plant. As noted in Section 3.7.1.1.1, the evaluation of slope stability is performed for the seismic level of the site-specific GMRS.

2.5.6 References

1. NUREG/CR-6372, “Guidance for Performing Probabilistic Seismic Hazard Analysis for a Nuclear Plant Site: Example Application to the Southeastern United States,” U.S. Nuclear Regulatory Commission, 2002.

2. ASCE 4-98, “Seismic Analysis of Safety-Related Nuclear Structures and Commentary,” American Society of Civil Engineers, September 1986.

3. ACI 318-2005, “Building Code Requirements for Structural Concrete and Commentary,” ACI Committee 318, American Concrete Institute, 2005.

4. NUREG/CR-0693, “Seismic Input and Soil–Structure Interaction,” Final Report, U.S. Nuclear Regulatory Commission, January 1979.