ENCLOSURE 1

M220064

Licensing Topical Report

NEDO-33914-A, Revision 2,
BWRX-300 Advanced Civil Construction and Design Approach

Non-Proprietary Information

GE Hitachi Nuclear Energy

NEDO-33914-A
Revision 2
June 2022

Non-Proprietary Information

Licensing Topical Report

BWRX-300 Advanced Civil Construction and Design Approach

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UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

April 27, 2022

Ms. Michelle Catts
Senior Vice President, Nuclear Programs
GE-Hitachi Nuclear Energy Americas, LLC
P.O. Box 780, M/C A-18
Wilmington, NC 28402

SUBJECT: FINAL SAFETY EVALUATION FOR GE-HITACHI LICENSING TOPICAL REPORT NEDO-33914, REVISION 1, "BWRX-300 ADVANCED CIVIL CONSTRUCTION AND DESIGN APPROACH" (DOCKET NO. 99900003)

Dear Ms. Catts:

By letter dated January 20, 2021, GE-Hitachi Nuclear Energy Americas, LLC (GEH) submitted Licensing Topical Report (LTR) NEDO-33914, "BWRX-300 Advanced Civil Construction and Design Approach," Revision 0 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML21020A135), to the U.S. Nuclear Regulatory Commission (NRC) staff for review and approval in support of a future licensing application for the GEH small modular reactor (SMR) under Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, "Domestic Licensing of Production and Utilization Facilities," or Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants." Subsequently, during the staff review, GEH submitted Revision 1 of the LTR by letter dated November 18, 2021 (ADAMS Accession No. ML21322A214).

The NRC staff has found GEH LTR NEDO-33914, Revision 1, to be acceptable for referencing in licensing applications for the GEH SMR design to the extent specified in the enclosed final safety evaluation (SE) that defines the basis for acceptance. In addition, on April 21, 2022 (ADAMS Accession No. ML22105A106), the Advisory Committee on Reactor Safeguards concluded that the staff's SE for LTR NEDO-33914, Revision 1, with the limitations and conditions imposed, is appropriate and should be issued. Therefore, the staff did not make any changes to the staff Advanced Final Safety Evaluation dated February 15, 2022 (ADAMS Accession No. ML22020A036).

In accordance with the guidance provided on the NRC's LTR website (http://www.nrc.gov/about-nrc/regulatory/licensing/topical-reports.html), the NRC requests that GEH publish an accepted version of this LTR within three months of receipt of this letter. The accepted version shall incorporate this letter and the enclosed SE after the title page. Also, it must contain in its appendices historical review information, such as requests for additional information, accepted responses, and the actual revised pages (showing revision bars) that were included as part of LTR NEDO-33914, Revision 1. The accepted version shall include an "-A" (designated accepted) following the report identification symbol.

If the NRC's criteria or regulations change so that its conclusion in this letter (that the LTR is acceptable) is invalidated, GEH and/or an applicant referencing the LTR will be expected to revise and resubmit its respective documentation or submit justification for the continued applicability of the LTR without revision of the respective documentation.

If you have any questions or comments concerning this matter, I can be reached via e-mail at Alina.Schiller@nrc.gov.

Sincerely,

/RA/

Alina Schiller, Project Manager
New Reactor Licensing Branch
Division of New and Renewed Licenses
Office of Nuclear Reactor Regulation

Docket No.: 99900003

Enclosure:
Safety Evaluation

cc: GEH BWRX-300 NEDC-33914 ListServ

SUBJECT: FINAL SAFETY EVALUATION FOR GE-HITACHI LICENSING TOPICAL REPORT NEDO-33914, REVISION 1, "BWRX-300 ADVANCED CIVIL CONSTRUCTION AND DESIGN APPROACH" (DOCKET NO. 99900003) DATED: APRIL 27, 2022

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LTR: ML22103A126
SE: ML22020A036 (Public) ***via e-mail** **NRR-106**

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SAFETY EVALUATION BY THE OFFICE OF NUCLEAR REACTOR REGULATION

LICENSING TOPICAL REPORT NEDO-33914, REVISION 1

BWRX-300 ADVANCED CIVIL CONSTRUCTION AND DESIGN APPROACH

GE-HITACHI NUCLEAR ENERGY AMERICAS, LLC

DOCKET NO. 99900003

1.0 INTRODUCTION

GE-Hitachi Nuclear Energy Americas, LLC (GEH), submitted Licensing Topical Report (LTR) NEDO–33914, Revision 0, "BWRX 300 Advanced Civil Construction and Design Approach," dated January 20, 2021 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML21020A137), as supplemented by Revision 1 (ADAMS Accession No. ML21322A214) dated November 18, 2021. The U.S. Nuclear Regulatory Commission (NRC) staff reviewed the LTR with respect to the provisions proposed by GEH for the advanced civil construction of the BWRX-300 small modular reactor (SMR).

In response to the NRC staff's requests for additional information (RAIs), GEH submitted letters dated August 19, 2021 (ADAMS Accession No. ML21231A255); September 13, 2021 (ADAMS Accession No. ML21256A008); and November 4, 2021 (ADAMS Accession No. ML21308A012). The NRC staff will evaluate the compliance of the final civil construction and design approach features for the BWRX-300 SMR during future licensing activities in accordance with Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, "Domestic Licensing Of Production And Utilization Facilities," and/or Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants," as applicable with the Limitations and Conditions (L&Cs) as outlined in Section 8.0 of this safety evaluation (SE).

1.1 <u>Purpose</u>

The purpose of the LTR is to provide guidelines for design, analysis, monitoring, and requirements for construction of a BWRX-300 SMR. The term "requirements," as used in the LTR, as well as is used in this staff SE, is not associated with any specific NRC regulation or NRC requirement unless specifically identified as such in this SE. The term is instead used to describe what GEH has proposed to use for construction of a BWRX-300 SMR using innovative and comprehensive approaches that ensure safe operation throughout the life of the plant. GEH has referenced NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR [light-water reactor] Edition" (SRP), Chapter 2, "Site Characteristics and Site Parameters," and Chapter 3, "Design of Structures, Components, Equipment, and Systems," as they apply to aspects of its proposed construction criteria. Because these SRP sections may not apply directly to the proposed construction of an embedded reactor, the applicant supplemented the SRP guidance with associated guidance from NUREG/CR-7193, "Evaluations of NRC Seismic-Structural Regulations and Regulatory Guidance, and Simulation-Evaluation Tools for Applicability to Small Modular Reactors," for the design of deeply embedded SMRs, as well as from associated industry standards and other guidance, as presented in its LTR and discussed in this SE. Specifically, the LTR describes the

criteria, methodologies, recommendations, and approaches specific to the BWRX-300 SMR design and construction, as discussed in item A-Q of LTR Section 1.1.

Implementation

An applicant who references a topical report in a licensing application must demonstrate that the application of the topical report to their specific facility is within the scope of the conditions in the topical report defining its application. The staff verifies relevant criteria for accepted-for-use topical reports during each licensing action to ensure that the topical report's conclusions are both valid and applicable to the particular licensing action under review.

Accordingly, upon implementation of this LTR into a site-specific application of the BWRX-300 design, the staff will evaluate each topical area designated below to ensure that each topic appropriately interfaces with the proposed license application to ensure consistency. The staff will also make its regulatory determinations regarding the topics discussed below, as applicable, during its review of any future license application.

1.2 Scope

The scope of the LTR includes the following:

- The specific regulatory basis for each methodology described in this LTR for the analysis, design, and construction of the BWRX-300 SMR.
- Guidelines and requirements for characterizing subsurface conditions, including geotechnical site investigations and laboratory testing programs, as well as the inspection and monitoring programs performed during the excavation, construction, and operation of the BWRX-300 SMR.
- Requirements and guidelines for performing foundation interface analysis (FIA) to ensure the stability of both structure and the in situ soil and/or rock during and after construction.
- Design requirements, acceptance criteria and guidelines provided in the LTR for the analysis and design of the deeply embedded reactor building (RB), including the development of site-specific geotechnical and seismic design parameters.
- The BWRX-300 SMR approach for addressing the interactions between the seismic Category I (SC-I) RB and the surrounding structures and foundations (II/I interactions).
- Generic seismic and geotechnical design parameters that ensure the applicability of the BWRX-300 SMR generic design for a range of conditions present at the majority of potential North American candidate sites.

1.3 Description of the BWRX-300 SMR

LTR Section 1.3 provides high-level information about the BWRX-300 SMR and its proposed construction techniques. The BWRX-300 SMR is a water-cooled, natural circulation-driven SMR with a power output of about 300 megawatts electric. GEH has described how the BWRX-300 basis for design includes nine previous generations of the boiling-water reactor (BWR) and

has evolved from the Economic Simplified Boiling-Water Reactor (ESBWR) Design Certification (DC), certified by the NRC in 2014 (10 CFR Part 52, Appendix E, "Design Certification Rule for the Economic Simplified Boiling-Water Reactor"). GEH has stated that the BWRX-300 incorporates design, analysis, and operating experience from the BWR operating fleet, advanced boiling-water reactor, and ESBWR, and adds evolutionary design improvements and new defense-in-depth design features and functions.

The BWRX-300 Reactor Pressure Vessel (RPV), Pressure Containment Vessel, and other important safety-related systems and components are located in the RB. The RB is placed in a vertical right-cylinder shaft and located below-grade to mitigate effects of possible external events, including aircraft crashes, adverse weather, flooding, fires, and earthquakes.

1.4 Reactor Building Below-Grade Shaft Construction

GEH proposes the open caisson technique as the preferred method to construct the RB shaft. A circular slurry shoring wall will be installed in the soil strata and socketed into the bedrock to stabilize the excavated shaft. The rock below the soil strata would be excavated to the bottom of basemat. Waterproofing material would be applied to the surface of the slurry wall and the rock face.

2.0 REGULATORY BASIS

In the LTR, GEH proposed innovative and comprehensive approaches to meet the NRC regulatory requirements of 10 CFR, Part 50, Appendix A, General Design Criteria (GDC). Specifically, requirements of GDC 1, “Quality standards and records,” and GDC 2, “Design bases for protection against natural phenomena.” Further, the construction approaches proposed by GEH in the LTR are established to meet the intent of NRC guidance, including the guidance prescribed in SRP Chapter 2 and SRP Chapter 3. Since there is no specific NRC guidance developed for embedded SMR reactors at this time, GEH used the guidance outlined in NUREG/CR–7193, as well as proposed construction requirements from industry standards.

LTR Section 2.0 provides statements of compliance with the regulations in 10 CFR Part 50 and 10 CFR Part 52, that GEH determined to be related to the civil construction and design of the BWRX-300 SMR. The LTR also identified design-specific information associated with relevant NRC guidance.

This LTR describes the intent to meet each of the relevant regulatory requirements for the BWRX-300 SMR. In some instances, the LTR indicates that specific design requirements for the BWRX-300 systems and components will be provided during future licensing activities.

The remainder of Section 2 describes each of the specific regulations that GEH addresses in this LTR. When the NRC receives an application for a BWRX-300 SMR, the staff will review the application against all applicable regulatory requirements related to the design and construction of the BWRX-300 SMR.

2.1 Regulatory Basis for Defining Site Subsurface Conditions

10 CFR Part 100

The regulations in 10 CFR Part 100 provide the reactor site criteria, including the physical characteristics, like seismology and geology, that shall be considered in siting a power reactor.

<u>10 CFR 100.20(c)(1)</u>

The regulation in 10 CFR 100.20(c)(1) points to 10 CFR 100.23. Part 100.20(c) requires that the Commission consider physical characteristics of the site.

<u>10 CFR 100.23</u>

The regulation in 10 CFR 100.23 sets forth the principal geologic and seismic considerations that guide the Commission in its evaluation of the suitability of a proposed site and adequacy of the design bases established in consideration of the geologic and seismic characteristics of the proposed site, such that, there is a reasonable assurance that a nuclear power plant can be constructed and operated at the proposed site without undue risk to the health and safety of the public.

2.2 Regulatory Basis for Development of Site Design Parameters

<u>10 CFR Part 50, Appendix A, General Design Criteria 2</u>

The regulation in 10 CFR Part 50, Appendix A, GDC 2 requires that nuclear power plant structures, systems, and components (SSCs) important to safety be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. This LTR describes a method for determining that a BWRX-300 SMR is designed to meet the requirements of GDC 2 for earthquake ground motions. The LTR focus on this hazard and does not discuss the other hazards in GDC 2, except for extreme winds, which is discussed in Section 6.3. As such, this SE does not discuss the other hazards.

<u>10 CFR 100.23(d)(1)</u>

10 CFR 100.23(d)(1) provides the requirements for defining the safe-shutdown earthquake (SSE) ground motion for a site and the need to address uncertainties in the site investigation and determination of site hazard.

2.3 Seismic Analysis Regulations

10 CFR Part 50, Appendix S, “Earthquake Engineering Criteria for Nuclear Power Plants,” requires that SSCs shall be designed to withstand the effects of the SSE ground motion or surface deformation are those necessary to assure: (1) the integrity of the reactor coolant pressure boundary; (2) the capability to shut down the reactor and maintain it in a safe-shutdown conditions; and (3) the capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 50.34(a)(1).

2.4 II/I Interaction Guidance

SRP Section 3.7.2, “Seismic System Analysis,” Revision 4, December 2013 (ADAMS Accession No. ML13198A239), Subsection II.8 provides three interaction criteria for a non-seismic Category I structure to a SC-I structure (II/I interactions). Each non-seismic structure should meet at least one of these criteria:

A. The collapse of the non-seismic Category I structure will not cause the non-seismic Category I structure to strike a seismic Category I SSC.

B. The collapse of the non-seismic Category I structure will not impair the integrity of seismic Category I SSCs, nor result in incapacitating injury to control room occupants.

C. The non-seismic Category I structure is analyzed and designed to prevent its failure under SSE conditions.

These criteria ensure that collapse of a non-seismic Category I structure would be acceptable and no physical interaction between the non-seismic Category I and SC-I structures take place in a site SSE.

2.5 Testing, Inspection, and Monitoring Regulations

GDC 1 requires that structures important to safety be constructed and tested to quality standards commensurate with the importance of the safety functions to be performed. LTR Section 3 addresses the investigations, testing, inspections and monitoring programs proposed for the BWRX-300 SMR.

3.0 INVESTIGATIONS, TESTING, INSPECTION, AND MONITORING PROGRAMS

Since the BWRX-300 RB structure would be deeply embedded, additional site investigations, laboratory testing, and field monitoring programs would be needed in addition to inspections during construction. Changes in in situ stress distribution in the subgrade materials during excavation and construction can induce movement in the surrounding soil and rock media.

GEH states that the suitability of a particular site would be verified through an extensive site investigation program. The laboratory and field testing programs would include both soil and rock properties including the properties of different interfaces. Parameters needed to model the interfaces are shown in LTR Figure 4-2 (also discussed in response to NRC RAI 02.05.04-01 dated November 4, 2021). A site monitoring program would monitor the conditions within the RB shaft and its surrounding media. Movements of the subgrade materials and change in groundwater would be particularly monitored due to their influence on the stability of the RB shaft. Necessary field instrumentation will be deployed to measure such changes. Additionally, the observations in the field monitoring program would be used to calibrate the FIA model discussed in LTR Section 4.0.

The staff notes that the scope of field investigations, laboratory and field testing, and field monitoring programs may be more than in a conventional LWR because the BWRX-300 SMR RB will be deeply embedded. In addition, GEH proposes to calibrate the FIA model with the observations made at the site. This calibration program, as discussed in the LTR and evaluated by the staff below, is acceptable to the staff.

3.1 Site Investigation and Subsurface Material Testing Programs

An extensive site investigation and testing of subsurface materials would ascertain whether a particular site is suitable for deploying the BWRX-300 SMR. A significant portion of the RB structure is deeply embedded in the subgrade materials, which may be comprised of soil, rock, or both. The interaction of the reactor structure with surrounding soil/rock media is important for the integrity of the RB structure and its response under both static and dynamic loads. Change

in the in situ stress field during excavation, construction, and operation of the BWRX-300 SMR may induce movement in the surrounding medium.

The site investigation and testing programs of the subsurface materials should be able to determine the necessary parameters of all models that would predict the response of the RB shaft and its surrounding media. GEH has developed the site investigation and field and laboratory testing programs following Regulatory Guide (RG) 1.132, "Geologic and Geotechnical Site Characterization Investigations for Nuclear Power Plants," Revision 3, October 2021 (ADAMS Accession No. ML21298A054) and RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, December 2014 (ADAMS Accession No. ML14289A600). In addition, the technical bases described in RG 1.132 are provided in Appendices A and B of NUREG/CR–5738, "Field Investigations for Foundations of Nuclear Power Facilities," November 1999 (ADAMS Accession No. ML003726925).

GEH states that it adheres to the guidance in NUREG/CR–5738 to develop the site characterization program for the BWRX-300 SMR. However, because the BWRX-300 SMR includes a deeply embedded RB, the site investigation and subsurface materials testing program would go beyond the current regulatory guidance of RG 1.132, RG 1.138, and NUREG/CR–5738.

The staff finds the approach GEH has presented is reasonable. The site investigation and programs for field and laboratory testing programs are developed following the current regulatory guidance and are supplemented by additional programs, as discussed further below, because the BWRX-300 SMR includes a deeply embedded RB structure.

The staff also notes that the site investigation program and the associated laboratory and site testing programs are somewhat dependent on the site and reactor design. The staff will perform a detailed evaluation to confirm that the final design features and associated analyses would satisfy the regulatory requirements of GDC 2 when the agency receives a license application for construction and or design of a BWRX-300 SMR.

3.1.1 Site Investigation Program

Figure 3-1 of the LTR shows a preliminary layout of the BWRX-300 SMR single unit plant and preliminary borehole locations for geotechnical and geophysical investigations for a typical site. Expected type and number of tests are given in Table 3-1. The LTR states in Section 3.1.1, that a minimum of 21 borings are anticipated for a BWRX-300 SMR installation, more than the recommended number of borings in Appendix D of RG 1.132 to ensure adequate characterization of the subsurface properties surrounding the RB structure. GEH has set the maximum required depth of these boreholes at approximately 120 m based on the change in in situ vertical stress.

The staff agrees that more boreholes compared to typical conventional nuclear plants will be necessary to characterize the surrounding media for BWRX-300 SMR because it is deeply embedded. However, the spatial variation of the subsurface characteristics and material properties greatly influence the minimum number of boreholes necessary for adequate characterization of the surrounding media. Because of the site-specific nature, the staff cannot make a determination at this time regarding the adequacy of the number of boreholes which would be necessary and the appropriateness of their specific locations with respect to the RB shaft. In addition, the actual dimensions of the RB and the in situ stress field would dictate

whether 120 m would be adequate for the maximum depth of the boreholes. Therefore, the staff also cannot determine whether the maximum depth of the borehole equal to 120 m would be adequate for every site. The number of boreholes and maximum depth are site-specific design requirements that will need to be provided during future licensing activities.

Additionally, GEH has stated, in LTR Section 3.1.1, a geological mapping program would map the rock fracture network in the same geological units exposed at nearby outcrops and in boreholes. In this context, the rock fractures include the geological discontinuities, such as rock joints, bedding planes, faults, zones of weakness, etc. The geological mapping program would characterize the rock mass with associated fractures for assessing stability of the surrounding media, development of the rock mass parameters, and guiding the field investigation and development of the FIA model, as discussed in LTR Section 4.0.

In response to NRC RAI 02.05.04-02, dated November 4, 2021, GEH has stated that it may construct additional inclined borings to intersect the geological discontinuities at better orientations and additional borings to investigate subsurface structural features (e.g., the fracture network) if the results from fracture mapping indicate the need. Data obtained from the borings would supplement the fracture network mapping leading to a better characterization of the rock mass. Surface geophysical measurements will also be used to better mapping of the subsurface features in between the borings. The borings will also provide recovered cores, televiewer measurements, seismic measurements, access for water pressure tests in bedrock, and allow installation of piezometers.

The staff agrees that the additional rock fracture network and the information from the rock fracture mapping can be used to assess the stability of the embedded shaft under both static and seismic loads. The staff also finds that the discussion on the rock fracture mapping program is consistent with Appendix B, “Geologic Mapping of Tunnels and Shafts,” of NUREG/CR–5738 and is, therefore, reasonable.

3.1.2 Laboratory Testing Program

The LTR lists the types of tests that will be conducted at a minimum in the laboratory on samples of soil and rock collected at the site to determine the required parameters needed in subsequent analysis, as described in LTR Section 4 and Section 5. GEH will perform a sufficient number of laboratory tests to minimize uncertainties in the geotechnical input properties of each soil and rock type. The estimated parameter values will be provided in terms of its mean and standard deviation. Estimates of systematic bias (epistemic uncertainty) and measurement bias (aleatory uncertainty) will be developed for the measured parameters.

In addition, GEH will conduct direct shear, triaxial strength, and other appropriate tests to estimate the strength and deformation properties of different types of interfaces, as discussed in LTR Section 4.0 and in response to NRC RAI 02.05.04-1, dated November 4, 2021. These interfaces include the interface between the RB structure and the surrounding rock/soil medium and interfaces between two geologic media (e.g., rock-rock interface for rock fractures, bedding planes, etc., and rock-soil interface). This testing program of an interface will determine the necessary parameters for the rheological model of an interface, as illustrated in LTR Figure 4-2. In response to NRC RAI 02.05.04-2 dated November 4, 2021, GEH has stated that large diameter samples containing the natural discontinuities present in the subsurface of a site would be used to determine the interface properties of the rock fractures using direct shear or triaxial tests (see LTR Section 3.1.3). GEH may also conduct other laboratory tests, such as

expansion, creep, erodibility, and durability, as needed, to characterize the subsurface materials.

The staff finds that the lists of laboratory tests for soil and rock properties are reasonable because these tests are typically performed in soil and rock engineering projects. In addition, the staff finds that the tests identified to determine strength and deformation parameters of different interfaces are appropriate and have been used in other industries, such as mining and construction industries. Strength and deformation properties of the interface between the RB structure and the surrounding medium are very important as they influence the response of the RB structure during an SSE. Characterizing the uncertainties associated with this and other interfaces will provide confidence in modeling the response of the RB structure and surrounding media, for example, in the FIA discussed in Section 4 of the LTR.

As discussed in LTR Section 4.3.4.3, the FIA modeling will use two sets of values for each of the elastic and/or inelastic properties measured during the loading and the unloading phases for both soil and rock media. Therefore, the staff expects to review material property values for both loading and unloading phases in a site-specific license application.

In addition, the staff will perform a detailed evaluation to confirm that the final design features and associated analyses would satisfy the regulatory requirements of GDC 2. The staff review will also include an assessment to determine if any other tests would be needed for a site-specific application for the BWRX-300 SMR. During any future license application review, the staff would verify whether all appropriate tests have been conducted to determine all the parameters necessary to design and construct the BWRX-300 SMR with an embedded RB at any designated site. Also, the staff would assess the plants structural safety performance for a site-specific SSE. A limitation and condition (L&C) # 1 for this testing program is described in Section 8.0 of this SE.

3.1.3 Characterization of Rock Mass Properties

GEH has proposed to use the empirical geomechanical rock mass classification systems, namely, Rock Mass Rating (RMR) system (Bieniawski, Z.T., “Rock Mechanics Design in Mining and Tunneling,” A.A. Balkema, 1984), and Geologic Strength Index (GSI) system (Hoek, E., and E.T. Brown, “The Hoek–Brown failure criterion and GSI – 2018 edition,” Journal of Rock Mechanics and Geotechnical Engineering, 11(3), pp. 445–463, 2019), to estimate the rock mass properties. These classification systems have several geologic and engineering parameters which are determined through rock fracture mapping at the site and laboratory testing. The staff notes that these rock mass classification systems are extensively used in mining, construction, and tunneling projects in a wide variety of rock masses. Therefore, the staff accepts these systems as reasonable to classify rock mass at a site selected to deploy a BWRX-300 SMR.

GEH has also stated that the presence of cavities would be identified during subsurface investigations of the site. The spacing and depth of investigation would be adjusted to detect the anticipated features, consistent with RG 1.132. The staff accepts the approach proposed by GEH as it is reasonable and generally practiced in industry.

3.2 Construction Inspection and Testing Program

3.2.1 Excavation and Foundation Inspections and Testing

GEH has proposed to conduct excavation and foundation inspections to satisfy the geotechnical and foundation inspection procedures contained in U.S. NRC, Inspection Procedure (IP) 88131, "Geotechnical/Foundation Activities," 2006 (ADAMS Accession No. ML060530176). Key site parameters would be verified through the average allowable static bearing capacity and maximum allowable dynamic bearing capacity for normal plus SSE loading. The staff considers the approach to verify key site parameters through inspections and testing of foundations as reasonable because they are consistent with the NRC inspection procedures.

3.2.2 Building Structure Construction Inspections and Testing

GEH has stated that the construction inspection and testing program would cover the project phases from the start of the shaft sinking through the BWRX-300 SMR construction and plant commissioning. GEH states that the construction inspection and testing program would satisfy the structural concrete inspection procedures of the U.S. NRC, IP 88132, "Structural Concrete Activities," 2006 (ADAMS Accession No. ML060530186) and structural welding inspection procedures of the U.S. NRC, IP 55100, "Structural Welding General Inspection Procedure," 1983 (ADAMS Accession No. ML061660235). The inspection program would include visual inspection of the accessible concrete surfaces and determination of susceptibility of concrete to deterioration. The staff concludes that GEH's proposed construction inspection and testing program is consistent with the procedures of the NRC IPs 88132 and 55100. In addition, the staff determines that the program also follows the guidance of the national standard, such as American Concrete Institute (ACI) 349.3R, "Evaluation of Existing Nuclear Safety-Related Concrete Structures," 2002 (Reapproved 2010) and American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code, Section XI "Rules for Inservice Inspection of Nuclear Power Plant Components," New York, NY, 2013.

3.2.2.1 Concrete Compressive Strength Testing Frequency

GEH has proposed a compressive strength testing program of safety-related concrete samples during construction. The in-process concrete strength would be tested in accordance with Section 5.6.2.1 of ACI 349-13, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," 2014, following the guidance in RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," Revision 3, May 2020 (ADAMS Accession No. ML20141L613). GEH has proposed additional sampling frequency requirements to ensure a statistically significant sample size. The staff finds the proposed approach to concrete strength testing to be reasonable because it is consistent with NRC guidance. Moreover, the proposed approach follows the national standard ACI 349–13, as suggested in RG 1.142. Additional sample testing to develop a statistically significant sample size is also reasonable. The staff notes that during the review of future licensing applications, the staff will audit the in-process concrete strength tests.

3.3 In-Service Monitoring Program

3.3.1 Scope of Structures Monitoring and Aging Management Program

The Structures Monitoring and Aging Management Program (SMAMP) of the BWRX-300 SMR would monitor the in-service conditions of the structures to detect any degradation and

deformation to ensure that credited safety functions as well as overall structural integrity are maintained throughout their design lives. This structural monitoring program begins after commissioning of the plant and continues until completion of plant decommissioning. The SMAMP of BWRX-300 SMR also includes below-grade structural members and foundations including monitoring of settlement and differential settlement. This SMAMP program, as evaluated by the staff in Section 3.3.2 of the SE, is reasonable. The staff notes that during the review of future licensing applications, the staff will audit the SMAMP and its implementation.

3.3.2 Framework of Structures Monitoring and Aging Management Program

The LTR has described the framework of the BWRX-300 SMAMP, which is based on the three-tier evaluation hierarchy given in ACI 349.3R, "Evaluation of Existing Nuclear Safety-Related Concrete Structures," 2002 (Reapproved 2010). The three-tier evaluation is shown schematically in LTR Figure 3-2. Following the guidance in ACI 349.3R, the LTR states that the inspection interval is defined in SMAMP and the personnel conducting the inspection shall be qualified as per Chapter 7, Qualification of Evaluation Team, of ACI 349.3R.

The LTR states that structural evaluation would follow the logic tree as shown in LTR Figure 3-2. The SMAMP has three tiers. In Tier 1, structures would be evaluated against qualitative and quantitative thresholds for visual inspections or condition surveys (First tier criteria) following Section 5.1 of ACI 349.3R. If a structure fails in Tier 1 evaluation, it would be subjected to Tier 2 evaluation. In Tier 2, structures would be evaluated against qualitative and quantitative thresholds for observed degradation in accordance with Section 5.2 of ACI 349.3R. Observed conditions failing the Tier 2 criteria would be evaluated to a Tier 3 evaluation. In Tier 3, structures would be evaluated using more enhanced methods to assess the structural condition. This evaluation would follow Section 5.3 of ACI 349.3R. Any corrective actions needed for a structure would follow the guidance given in Chapter 8, Repair, of ACI 349.3R. Provisions of the SMAMP would also include periodic sampling and testing of groundwater to assess whether the below-grade structures are exposed to an aggressive environment. If necessary, addition site-specific criteria would be developed.

The staff finds from the description of the SMAMP in the LTR that it would be designed following the national standard ACI 349.3R, and therefore the design of the program is reasonable. The definitions of the three tiers of evaluation follows ACI 349.3R. The logic tree for evaluation, as shown in Figure 3-2 of the LTR, is from Figure 5.1 of ACI 349.3R. The personnel conducting the evaluations shall be trained as per Chapter 7 of ACI 349.3R. Any remedial action needed would be in accordance with ACI 349.3R. The staff also finds that this section of the LTR is consistent with the provisions given in the latest version of ACI 349.3R, "Report on Evaluation and Repair of Existing Nuclear Safety-Related Concrete Structures," 2018. Based on the above, the staff finds that the description of the BWRX-300 SMAMP is reasonable as it is based on a recognized standard.

3.4 Field Instrumentation Plan

The LTR states that field instrumentation would be used to monitor the magnitude, spatial, and temporal distributions of the deformation and displacements of the media surrounding the deeply embedded BWRX-300 RB structure. In addition, distribution of the pore pressure would also be monitored using the field instrumentation. Short-term and long-term settlement monitoring plans are developed to measure both vertical and horizontal displacements in and around the structures. Differential distortion across the footprint of the foundations of the control building (CB), turbine building (TB), radwaste building (RwB), and RB, as well as the differential

settlement between any two structures, would be monitored. Locations of the sensors would depend on the areas anticipated to have a large response. Where practical, sensors would be connected to datalogger(s) for periodic measurements.

The site monitoring program of the subsurface materials surrounding the RB structure should be able to detect and quantify any changes or movements in the surrounding media so that appropriate actions can be taken at each stage of the life of the reactor. The staff finds this generic description of the field instrumentation plan to be reasonable as it has provisions to detect any deleterious movement of the media surrounding the reactor system. Any deleterious movement of the surrounding media would need rehabilitation to restore the stability. It should be noted that a field instrumentation plan should be both site-specific and design-specific. Therefore, the staff will review the instrumentation plan with details of the instrumentation to be deployed considering a specific site and details of the reactor design when the agency receives a site-specific application for the BWRX-300 SMR.

4.0 FOUNDATION INTERFACE ANALYSIS

In this section, the LTR discusses the FIA. The purpose of the FIA model is to ensure that the BWRX-300 RB, CB, TB, and RwB structures and the surrounding soil and/or rock remain stable throughout the life of the plant and meet the guidance in SRP Section 2.5.4, “Stability of Subsurface Materials and Foundations,” Revision 5, July 2014 (ADAMS Accession No. ML13311B744). The FIA will be conducted at different life stages of the BWRX-300 SMR to assess the construction plans, possible ground improvements, excavation support, and foundation interface design. The predicted foundation interface behavior would be compared with the actual observations made in the field monitoring program, as discussed in Subsection 3.4 of this LTR. Additionally, the results of the FIA would be used to verify the RB shaft design, as discussed in LTR Section 5.1.3.

LTR Section 3.4 provides the verification of the FIA results with the measured field observations. The field instrumentation approach, as described in Section 3.4, is used for monitoring and evaluating possible instabilities of the subgrade materials during the excavation, construction, and operation of BWRX-300 SMR. Together with the results of FIA, as described in Section 4.3.4, this approach is beyond the current guidance of SRP Section 2.5.4.

4.1 Foundation Interface Analysis Model

The FIA is a three-dimensional numerical model that examines the response of the BWRX-300 SMR’s structures and its surrounding media from change of in situ subgrade conditions at each life stage: excavation, construction, loading, and operation of the reactor. The numerical model will be calibrated with the measured response at the site at each life stage.

The FIA model has the capability to incorporate nonlinear stress-strain response of the soil and rock media, the interface to simulate slippage along and opening across an interface, limited structural systems, soil/rock support systems, and fluid-soil/rock interaction. An interface could be between the RB shaft and surrounding media (rock/soil), faults, rock joints, etc. The model has the capability to analyze the interaction of the BWRX-300 SMR RB structure and the surrounding media at different life stages of the RB structure.

The staff finds the description of the FIA numerical model to be reasonable because the description gives an overall picture of what the FIA model is intended to accomplish in addition to how the model results would be calibrated.

4.2 Subgrade Material Constitutive Models

The LTR states that the constitutive models of the BWRX-300 FIA would be based on characteristics of the site-specific data obtained from the field and laboratory testing programs, as described in LTR Section 3.1. Measurement results from the field instrumentation, described in LTR Section 3.4, would be used to modify the input parameters of the selected models. The staff finds this approach to be reasonable as it is generally used in the industry.

4.2.1 Soil Constitutive Models

GEH expects that a nonlinear constitutive model would be appropriate to represents the stress-strain relationship of the soils to be encountered at a site. GEH has proposed to use the bilinear Mohr-Coulomb model for representing the soil behavior. This model exhibits linear, elastic deformation with increasing stress up to the failure. Beyond failure, the stress-strain behavior is represented by a fully plastic model. Other more sophisticated constitutive relationships, e.g., strain-hardening or strain-softening models, may be used if the soil(s) at the site show(s) the need. The staff finds the discussion of selecting a particular constitutive model for representing the stress-strain behavior of site-specific soil(s) to be reasonable as it is generally used in the industry. The staff notes that during the review of future licensing applications, the staff will review the selected constitutive model to represent the soil response under load and the associated parameters.

4.2.2 Rock Constitutive Models

Rock mass may contain several types of fractures or discontinuities: rock joints, faults, bedding planes, etc. These fractures may make the rock mass an assemblage of complex-shaped blocks. In addition, the discontinuities are generally significantly weaker than the rock matrix (intact rock blocks) and mostly control the rock mass behavior. Response of a fractured rock mass may become complex from interaction among compression, translation, rotation, and potential generation of new or opening of existing fractures. The LTR mentions two constitutive models to represent the stress-strain response of the fractures rock mass: Mohr-Coulomb model and Generalized Hoek-Brown (GHB) 2018 model. The shear strength of a rock mass, represented as a Mohr-Coulomb material, are developed from the results of field investigation and laboratory tests, discussed before. The GHB model uses the uniaxial compressive strength of the intact rock and the GSI geomechanical classification scheme of the rock mass to estimate the rock mass strength and deformation parameters.

The staff notes that both Mohr-Coulomb and GHB models are commonly used in rock engineering applications. The staff agrees with GEH that GHB model may be more suited to represent a fractured rock mass as it has been developed specifically for that purpose incorporating the rock fracture network information of the rock mass through the GSI index value. The staff notes that during the review of future licensing applications, the staff will review the model selected to represent the response of a rock mass under load and determination of the model parameters.

4.3 Non-Linear Foundation Interface Analysis Approach

In this LTR Section, GEH highlighted the sections where the interface modeling, structural modeling, and fluid-soil interaction in the FIA model at different life stages of the BWRX-300 SMR (siting, excavation, and construction) are discussed. Interface modeling includes the

contact between the structure and the surrounding soil and/or rock media, and the contact between two sides of natural fractures or discontinuities, such as bedding planes, rock joints, fault planes, that are present in the rock mass.

4.3.1 Interface Models

4.3.1.1 Interface Between the Structures and the Subgrade Media

The LTR describes the response of the interface between the structure and the surrounding soil and/or rock media. The response of the interface or the contact plane between the structure and the subgrade media significantly affects the pressure exerted by the soil or rock medium on the structure. During a seismic event, the response of the structure with respect to the surrounding media would be controlled by the response of the wall interfaces. In comparison, the response of the interface at the base may not be that critical for the RB as deep embedment will likely control the sliding and overturning.

The LTR has provided the interface rheological model typically used with the BWRX-300 SMR in LTR Figure 4-2. Two sets of springs, one along and another across the interface, between two opposing sides of an interface are used as the interface element in the numerical model. The slippages along and opening/closing across the interface are controlled by these two springs. A series of these spring couplers would model the response of the entire interface. The relative sliding (shear) response of each spring along the interface is controlled by its stiffness. The LTR is proposing to use the elastoplastic Mohr-Coulomb criterion to model the shear behavior along the interface. The dilation/contraction or opening/closing of the interface is controlled by the normal stiffness of the spring and will be modeled using a tensile strength cut-off of the spring.

GEH has proposed that the parameters of the interface model, as shown in Figure 4-2 of the LTR, would be determined from the laboratory tests discussed in LTR Section 3.1.2. The LTR has also mentioned use of information collected in LTR Section 3.1.3, Characterization of Rock Mass Properties, to determine the interface parameters of the interface between the structure and the surrounding media. GEH, in response to the NRC Question 02.05.04-01, states that direct shear and/or triaxial test would be performed on rock discontinuity samples to estimate the interface properties.

In addition, GEH, in LTR Section 4.3.1.1 and in response to RAI 02.05.04-01, has stated that the development of the interface parameters should be consistent with the limitations and modeling guidance of the software used in the FIA. The staff finds the overall approach presented to estimate the parameters of the rheological model of the interface between the structure and the surrounding media by measuring it in laboratory experiments to be reasonable because it is commonly used in the industry. The staff notes that during the review of future licensing applications, the staff will review the interface model (Figure 4-2) parameters determined from the samples collected during site investigation. The staff notes, however, that if the laboratory-measured parameter values are outside the bounds of the selected software, then the software is not appropriate for the FIA modeling purpose for the scenario.

4.3.1.2 Fault or Joint Planes or Interfaces Between Bedding Units in a Geologic Formation

Faults, joints, or bedding planes are geological discontinuities of the rock mass. Response of these discontinuities in rock mass surrounding the BWRX-300 SMR’s structures are analyzed throughout the life stages of the facility. GEH proposes to model these discontinuities using the

approach discussed in LTR Section 4.3.1.1, Interfaces Between the Structures and the Subgrade Media. The parameters of the rheological model of a geological discontinuity, as shown in LTR Figure 4-2, will be determined through laboratory tests. Specifically, direct shear test and/or triaxial tests are conducted on recovered rock cores with natural discontinuities from field investigation, as discussed in response to RAI Question 02.05.04-01. Additionally, if properties of specific discontinuities are required, samples of these discontinuities would be collected in the field for testing in the laboratory. In response to RAI Question 02.05.04-01, GEH is also proposing to conduct laboratory tests on artificial surfaces. It is not clear how artificially created interfaces can be substituted instead of natural discontinuities because the roughness structure of both types of surfaces can be fundamentally different. The staff will review in a future site-specific application the rationale of using artificially created interfaces to determine the parameters of natural rock discontinuities.

Additionally, GEH proposes to use the weakest strength parameters of the interface elements out of the results from strength tests conducted on multiple discontinuities. The interface would have reduced strength after some displacement (slip). Strength reduction from the peak strength may be accomplished using strength reduction factors to estimate these residual strength parameters. In the response to RAI Question 02.05.04-01, GEH proposes to use the minimum parameter values representing the residual state of the discontinuities present in the rock mass.

The staff agrees with GEH that to model the natural discontinuities using the rheological model shown in LTR Figure 4-2 is reasonable because this rheological model has been used in numerous projects around the world and is commonly used in the mining and construction industries. The staff finds the proposal to estimate the model parameters from laboratory testing of samples of natural discontinuities, collected during field investigation at the site, reasonable as the parameters would be measured directly from samples containing actual discontinuities. The staff also agrees that the residual strength parameters of a natural discontinuity after small slippage would be smaller than the peak values. Strength reduction from the peak state to the residual state of the natural discontinuities is appropriate; however, the staff concludes that the minimum peak values of a set of discontinuities represent the peak value of the weakest discontinuity. The staff notes that in future licensing applications, the staff will review the parameters of the rheological model (Figure 4-2) of natural discontinuities from the laboratory tests. This review will address, as discussed above, the following items: (1) reduction of peak shear strength to residual shear strength of a discontinuity and (2) the minimum peak shear strength that is the shear strength of the weakest discontinuity in the discontinuity set.

4.3.2 Structural Elements Representation in the Foundation Interface Analysis Model

The LTR states that the FIA model of the BWRX-300 SMR will include representations of the RB structure and soil stabilization elements, such as liners, stabilization walls, rock anchors, etc. They will be represented as linear elastic materials in the model to capture the interaction between the structure and the subsurface materials because the surrounding materials will fail long before the onset of structural failure. Only elastic response is assumed to determine whether deformations or stresses in any structural member reach levels beyond the intent of the design. GEH also states that the model of the RB structure would be sufficiently refined to adequately capture the interaction with the surrounding media and transfer the loads to and from the media.

The staff finds that the use of linear elastic materials properties of the RB structure and stabilization elements is reasonable because the surrounding media would undergo plastic and large deformation before the structural members show onset of inelastic behavior. Additionally, the staff finds the use of only elastic properties is reasonable to identify structural members experiencing undesirable levels of stresses or deformations.

4.3.3 Fluid-Soil Interaction

GEH proposes to measure the elevation of the groundwater table and hydraulic properties of the subsurface materials during site investigation. The FIA model may include a hydraulic interface to simulate the effects of groundwater on the structure during the life stages of the BWRX-300 SMR structures. The model would be capable of simulating both short-term and long-term effects of dewatering.

The staff notes that the proposed approach only considers matrix flow of the groundwater. Depending on the fracture network in the rock layers and level of groundwater at the site, groundwater may flow into the excavation through selected discrete fractures (fracture flow). The staff also notes that occurrence of fracture flow is site dependent and will conduct an appropriate review as part of a site-specific license application for construction and or design of a BWRX-300 SMR. Additionally, the staff notes that the proposed model is not capable of simulating any deleterious effects from a corrosive environment that the presence of water may introduce.

4.3.4 Analysis Staging Approach

GEH states that the FIA is conducted at different life stages of the BWRX-300 SMR structures to determine the stress and deformation at different points of interest in the numerical model of the structures and the surrounding media.

4.3.4.1 Site Characterization

The FIA model at the site characterization stage of the BWRX-300 SMR system would simulate the initial in situ stress field. This stress field will be aligned with the initial baseline displacement field. The staff notes that GEH has stated that the initial stresses will include the measured horizontal stresses and any influence of the groundwater, if applicable. The staff agrees with the approach as this is commonly used in modeling an excavation. The staff also notes that the initial stress field should include vertical and horizontal stresses, the vertical stress at a site is generally from the gravity driven load. In response to NRC RAI Question 02.05.04-08, dated November 4, 2021, GEH clarifies that the in situ stress field (vertical and two horizontal stresses) will be measured at the site.

4.3.4.2 Excavation

During excavation of the shaft to place the RB structure, tensile stress may develop in the surrounding media due to redistribution of the initial stresses. The change in in situ conditions will be modeled in the FIA model during excavation of the shaft. The progression of excavation will be simulated by removing layers of soil or rock within the shaft in the FIA model. Stability of the shaft as the excavation progresses will be verified by comparing the FIA results with the actual field observations.

The staff finds the proposed modeling scheme during the excavation stage described in the LTR to be reasonable as it is typically followed in modeling excavations in soil/rock medium. Comparing model results with the actual observations will allow calibration of the FIA model parameters as the excavation progresses.

4.3.4.3 Construction

Construction of the reactor adds additional loads to the surrounding media. GEH would use in the FIA model loading elastic and/or inelastic properties of soil and rock in place of unloading properties used in analyses of the Site Characterization (Section 4.3.4.1) and the Excavation (LTR Section 4.3.4.2) stages. GEH proposes to compare the field observations from the field monitoring (Section 3.4) and construction inspection (Section 3.2). Effects of any soil movement or displacement along joints will be continually analyzed to assess stability of the structure, foundation, and surrounding media.

The staff finds that the modeling approach during the reactor construction stage is reasonable because it is typically followed in the industry. Comparing the model results with field observations and construction inspection would further calibrate the FIA model. In addition, effects of any actions can be easily investigated before implementation.

4.3.4.4 Loading

Construction of civil structures and foundations, in addition to placement of mechanical and electrical components, introduces permanent dead loads to the reactor system and the surrounding media. Other loads, such as weight of the fuel, water in the pools, cranes, and other permanent loads would be introduced in the FIA model at this stage. In addition, loads from the foundations of the CB, TB, and RwB structures would also be introduced in the model at this stage together with the loads from any backfill materials. Comparison of the FIA results with the observations from monitoring will continue at this stage.

The staff finds that the approach described at this loading stage is reasonable as it is typically followed in the industry. Comparison with the monitored observations during the phases of construction and operation would be expected for the BWRX-300 SMR (LTR Figure 4.8).

4.3.4.5 Start-Up and Operation

During the operational life, the BWRX-300 SMR may experience additional loads from seismic ground motion, floods, and any potential subsurface instability. The FIA model would be used to assess the potential effects of these additional loads on the environment of the reactor. In addition, GEH proposes to use the FIA model to assess the response in between points monitored by the field instrumentation.

The staff finds the proposed approach is reasonable because it is typically used in the industry. In addition, use of the modeling results to assess the response at points without any actual observations is also common industry practice and would be expected for the BWRX-300 SMR construction and operation.

5.0 DESIGN ANALYSIS

5.1 One-Step Design Analysis Approach

In LTR Section 5.1, GEH presents the overall approach to the analysis of the RB under the effects of the imposed static and dynamic loads, which includes:

1. Self-inertia loads including loads from equipment and pool water,
2. The mass and impedance of the surrounding in situ subgrade materials,
3. Groundwater hydrostatic pressure; and
4. Overburden loads and the interaction with the surrounding RwB, CB, and TB foundations and structures.

In this one-step approach, GEH proposes to implement the process laid out in Section 3.1.2 of American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) 4-16, “Seismic Analysis of Safety-Related Nuclear Structures and Commentary,” 2017, for the design of the BWRX-300 RB structure. Static and dynamic structural stress demands are obtained directly from the results of the soil-structure-interaction (SSI) analyses of combined models that include the finite element (FE) representations of the RB structure and the surrounding medium. The surrounding subgrade is represented by a layered half-space continuum with equivalent linear elastic stiffness properties and complex damping.

Stress demands on the members of the RB structure due to static earth pressure, structural self-weight, equipment weight, and live loads are calculated by applying 1g gravity loads on the combined model of the RB structure and the subgrade continuum. The structural demands due to overburden pressures from the nearby foundations are also calculated by the 1g static analysis. Additional static analyses are performed to calculate the structural demands due to hydrostatic wall pressures from the pool water, and normal operating and accidental pressure loads. Separate analyses provide the structural demands due to normal operating and accidental pressure and thermal loads. Structural demands due to seismic inertia loads and dynamic soil/rock pressure loads are obtained from the seismic SSI analyses, described in LTR Section 5.3.

The methodology used for development of RB FE model is based on the methodology described in LTR Section 5.1.1 and the SSI modeling assumptions presented in LTR Section 5.1.2. Equivalent linear properties are used as input for the static and seismic SSI analyses developed as described in LTR Sections 5.2.1 and 5.2.4, respectively. LTR Section 5.1.3 presents the unique BWRX-300 SMR approach used to demonstrate that the linear elastic SSI analyses provide soil and rock pressure load demands with sufficient design load margins to address the modeling uncertainties.

The staff finds that the overall approach to compute the design demands of the RB is reasonable because GEH has committed to the use of consensus practices of ASCE 4 and other static analysis methods.

5.1.1 FE Model of RB Structure

The staff has reviewed the modeling approach of the RB structure, presented in LTR Section 5.1.1, which states that the structural FE model will consist of beam, shell, solid, and spring elements representing the RB structural configuration for all main structural members including shear walls. The FE model includes gross discontinuities such as large openings and member eccentricities. Rigid beams or rigid links are proposed for modeling member eccentricities and offsets. Linear elastic contact springs connect the RB structural and subgrade FE models. Stiffness properties, which are assigned to the contact springs, represent the interaction mechanism between the structure, the water proofing material, and the soil/rock.

The LTR states that the mesh of the FE models will be sufficiently refined to produce stress demand calculations that are not significantly affected by a further refinement of the FE size or shape. Finer meshes will be used around penetrations and openings that are larger than half of the wall or slab thickness. Meshes of major walls and slabs to consists of at least four shell elements along the short direction and at least six shell elements along the long direction.

The FE models used for seismic SSI analyses will be sufficiently refined mesh to transmit the entire frequency range of interest for the seismic design of the RB SSCs, in accordance with ASCE/SEI 4-16, Section 5.3.4. The LTR states that the material properties of the concrete structural elements would be based on ACI 349-13 and of steel or steel-plated composite (SC) members of the RB and would be based on American National Standards Institute (ANSI)/American Institute of Steel Construction (AISC) N690-18," Specification for Safety-Related Steel Structures for Nuclear Facilities," 2018, respectively.

The staff finds the LTR approach to modeling the RB structure reasonable because such modeling practices have been used in other nuclear installations and have provided acceptable results.

5.1.2 Soil-Structure Interaction Modeling Assumptions

In LTR Section 5.1.2, GEH presents the approach to assigning the stiffness properties established using the approaches discussed in LTR Sections 5.2.1.1 and 5.2.1.2. The contact springs represent the interface between the structure, the waterproofing material, and the surrounding soil/rock mass. The LTR states the upper bound stiffness properties are assigned to the contact springs normal to the RB exterior walls. Contact springs in the vertical and tangential direction are assigned very low stiffness values to simulate the effect of zero friction between the wall and subsurface in the respective directions.

The soil and rock strata in the SSI models use isotropic linear elastic properties and are used to compute the design demand for the RB structure. Potential discontinuities or cavities in the rock mass are not explicitly included the SSI models. The LTR has made simplifying assumptions to support the above approach: (1) the properties of the subgrade materials are linear elastic (small strain), (2) nonlinearities at the soil/rock interfaces with the structures are neglected, (3) the rock mass is continuous free of any discontinuities, and (4) the static lateral pressures on the RB shaft due to the weight of self-supporting rock can be neglected. Assessment of the potential effects of waterproofing materials and RB shaft construction have been discussed in LTR Section 5.3.8.

The staff finds the assumptions made for assigning the upper-bound stiffness in the normal direction and a low value of stiffness in the vertical and tangential directions to the springs at the

interface of the RB and the surrounding media to be reasonable because they are consistent with current practice used in SSI analysis.

The design of the BWRX-300 SMR inherently assumes a self-supporting or stable rock mass surrounding the RB shaft, as indicated in this section of the LTR. GEH, in response to RAI 02.05.04-05 and 02.05.04-06, states that a stable shaft excavation would have no unstable blocks in its surrounding that may slide into the excavation or potentially unstable blocks would be stabilized by reinforcement. A self-supported (even with some temporary reinforcement) excavation would be needed to place the RB and to estimate the earth pressure loads to be considered in the generic design of the RB structure. GEH does not consider a rock mass stable over the life of the BWRX-300 SMR if it requires permanent supports (reinforcement), as stated in response to NRC Question 02.05.04-06. As discussed in its response, the "BWRX-300 SMR can be deployed at soil sites and sites having rock masses that require support during the excavation and construction of the deeply embedded" RB shaft. GEH stated that "The as-built site-specific subgrade conditions must ensure the stability of the BWRX-300 SMR power block foundations." Although small yielding is expected even in a self-supported rock mass, the yielding should not induce earth pressure exceeding the RB design limit. GEH proposes to identify fractures zones, joints, bedding planes, discontinuities, and other zones of weakness at a site through characterization of the rock fracture network (response to RAI 02.05.04-02) to analyze stability of rock mass surrounding the RB shaft.

GEH proposes to estimate the earth pressure on the RB shaft from unstable rock blocks in the surrounding rock mass using the FIA that includes rock-rock discontinuities with appropriate properties (LTR Section 4.3.1.2 and response to RAI 02.05.04-01 and 02.05.04-05) and/or force-equilibrium analysis. The staff notes that the force-equilibrium method implicitly assumes rigid blocks. As such, the rationale for GEH's assumption that rigid blocks would be appropriate is a site-specific design requirement that will need to be provided during future licensing activities.

The primary focus in estimating the earth pressure will be to include all unstable blocks so that the estimated earth pressure will include contribution from all unstable blocks and will be bounding (response to NRC Question 02.05.04-05). If the subsurface conditions at the site result in large loads to the RB shaft, the BWRX-300 SMR may still be sited with additional mitigating measures, such as over-excavating and backfilling, to reduce the load on the RB shaft.

Based on the above discussion, the staff finds that assumption of a stable rock mass surrounding the RB shaft is reasonable. The staff will make its final conclusions on the methods used to identify unstable blocks in the surrounding rock mass and the mitigation measures to be taken to stabilize them during any future site-specific license review of the BWRX-300 SMR. A L&C # 2 is described in Section 8.0 of this SE.

5.1.3 Design Earth Pressure Load Validation

In LTR Section 5.1.3, GEH proposes to compare the estimated soil and rock pressure loads on the exterior walls of the RB structure using the results of the nonlinear FIA analysis and the linear elastic 1g design analysis to assess: (1) the effects of nonlinear and possibly anisotropic subgrade response on the soil and rock pressures, and (2) the conservatism of the soil and rock pressure loads to design the RB structure from the 1g analysis, as described in LTR Section 5.1.2. Only unimproved soil and rock conditions would be considered due to uncertain longevity of any ground improvements made at the construction stage. The rock pressure on

the RB shaft wall may be uniform because of contact grouting. Alternatively, it can be concentrated loads if rock blocks are reinforced to stabilize the surrounding rock mass. GEH has assumed that the excavation is stable with the initial (or, temporary) rock support. In addition, the liner would be able to withstand the entire rock load as the rock support systems placed initially would degrade over time. Additional load may come from presence of hydrostatic head and swelling of the rock. In addition, loads from other surface structures may be transferred to potentially unstable rock blocks and, thereby, impart at least a fraction of the load on the RB structure.

The staff finds that GEH has proposed a reasonable approach to develop an estimate of the earth pressure comprising of soil and/rock pressure by comparing the results from the FIA model and 1g design analysis because this approach is commonly used in the nuclear industry.

The distribution of the lateral pressure with depth at the site obtained from the FIA model is compared with that from the 1g design analysis to establish the load margins. If the calculated soil and rock design load margins are below a threshold established to adequately address the uncertainties and variabilities of subgrade properties, GEH would adjust the linear soil and rock stiffness properties used in 1g design analysis.

In addition, for sites in high seismic regions, if the FIA results indicate that nonlinearity of geometry and material properties introduces significant anisotropic effect on estimating the rock and soil pressures, GEH proposes to conduct sensitivity analyses of the nonlinear SSI model. The sensitivity analyses are expected to assess the effects of nonlinear soil and rock response in developing the demand of dynamic lateral pressure.

The staff finds that GEH has presented a reasonable approach to estimate the lateral loads at a site. Comparing the results from the FIA model with the 1g linear model is expected to develop a set of material properties that would bound the results from the FIA model. In cases with high seismicity and/or highly nonlinear subgrade materials, GEH would conduct sensitivity analyses to assess the effects of nonlinearity and high seismicity. The staff notes that during the review of future licensing applications, the staff will review the computed lateral earth pressure loads on the exterior wall of the RB structure as well as the need for nonlinear SSI analysis.

5.1.4 Probabilistic Earth Pressure Analyses

GEH has proposed to perform probabilistic assessment of the earth pressure used to design the deeply embedded RB shaft. The probability density function of the subgrade pressure would be computed at the discrete regions of the external wall of the RB structure in contact with soil or rock medium. The objective of this assessment would be to demonstrate that the magnitude of earth pressures used would adequately bound the uncertainties in calculating the earth pressure loads. Two types of uncertainties would be addressed: (1) epistemic or uncertainties associated with the models used to estimate the earth pressure, and (2) aleatory or uncertainties of the parameters posed by natural randomness and uncertainties in measuring the properties of the subgrade materials.

The staff finds that GEH has presented a reasonable approach to characterize uncertainties associated with the measured parameters and models for estimating the earth pressure, as reviewed below. Use of the probability density function of the subgrade pressure is the standard approach in statistical analysis to characterize the types of uncertainties.

5.1.4.1 First Order Second Moment Method

GEH has proposed use of the First Order Second Moment (FOSM) method to estimate the probability density function of the ground pressure. The mean and variance of the earth pressure are calculated using the simplified models in LTR Section 5.1.4.3 or from the results of the FIA model, discussed in LTR Section 4. The derivatives of the earth pressure with each parameter are calculated for each discretized region.

The staff finds that GEH has presented a reasonable approach to use the FOSM method to calculate the mean and variance of the earth pressure at the discretized region as this method is widely used in probabilistic analysis. The staff also notes, as GEH has described in this section, that the FOSM method is appropriate if the relationship between the ground response and the geotechnical parameters are generally linear. For highly nonlinear cases, higher order formulations or the Monte Carlo method (discussed in LTR Section 5.1.4.2) would be appropriate to use.

5.1.4.2 Monte Carlo Method

GEH has proposed to use the Monte Carlo method as an alternative to the FOSM method to address the uncertainty associated with the probability distribution of the earth pressure. A minimum of 60 random realizations would be generated of each parameter whose variation has important effects on the earth pressure. These generated parameters are then used to calculate the earth pressure including its distribution.

The staff finds that GEH has reasonably selected the Monte Carlo method to assess the variation of the estimated earth pressure. The Monte Carlo method is widely used to propagate the uncertainties in the parameters if input uncertainties are represented as distributions, as discussed above and in NUREG/CR–2300, “A Guide to the Performance of Probabilistic Risk Assessments for Nuclear Power Plants,” Volumes 1 and 2, 1983 (ADAMS Accession No. ML063560439 and ML063560440), Section 12.4.3.1.2, Monte Carlo Simulation. Additionally, the Monte Carlo method is also useful if the relationship between the parameters and the response is highly nonlinear.

5.1.4.3 Probabilistic Analysis Earth Pressure Models

In this section, GEH has discussed methods to estimate the variation of earth pressure accounting for variation of individual parameters. Each parameter, whose variation has a significant effect on the estimated earth pressure, is related to the earth pressure in each discretized region through: (1) an analytical model, (2) a force-equilibrium model, or (3) a FE or a finite difference model. Analytical models give the distribution of earth pressure calculated using the individual distributions of subgrade material properties. Force-equilibrium models may be used to assess stability of individual rock blocks by analyzing the potential to slide along the discontinuities.

The staff notes that GEH has presented three alternative methods to estimate the variation of the earth pressure due to variation of individual parameters. All these methods are commonly used depending on the scenario to be analyzed. The staff also notes that although the force-equilibrium method, as shown in LTR Figure 5-1 for a sliding wedge, can be used to estimate the probability of sliding given the distribution representing each parameter, the method implicitly assumes rigid blocks. Stresses and strains cannot develop in these blocks. In

contrast, both the FIA model (discussed in LTR Section 4) and the design analyses (discussed in LTR Section 5) deal with deformable blocks as the blocks generate stresses and strains.

5.1.4.4 Combining Discrete Probability Distributions

GEH has discussed the approach to develop a continuous earth pressure distribution by combining different discrete distributions. GEH proposes to use the Monte Carlo method for combining these discrete parameter distributions with their degree of belief for a particular outcome. GEH wants to determine the belief probabilities using “a lottery or a probability wheel.”

The staff finds that GEH has presented a reasonable approach to combine different occurrence distributions into a continuous combined distribution of the final outcome or parameter (e.g., estimated earth pressure). This approach is typically used in probabilistic analysis. GEH also proposes to use the degree of belief to assign the probability that the results for each random realization would belong to a particular process or model. This is a reasonable approach; however, the staff notes that during the review of a site-specific license application, the staff will audit the values of the degree of belief have been determined.

5.2 Site-Specific Geotechnical and Seismic Design Parameters

GEH describes their approach to develop the equivalent linear properties of the soil and/or rock masses surrounding the RB shaft used in static SSI analysis in LTR Section 5.2.1. Approaches proposed to develop the magnitude and frequency content of the site-specific design ground motion spectra are discussed in LTR Section 5.2.2. Probabilistic site response analyses (SRA) to be conducted to incorporate the effects of overlying materials with appropriate epistemic uncertainties and aleatory variabilities are also discussed in Section 5.2.2.

Five sets of ground motion time histories compatible with the ground motion design spectra have been generated and described in Section 5.2.3. Results of these SRAs would be used to develop the stiffness and damping properties of the subgrade materials, as discussed in LTR Section 5.2.4.

5.2.1 Equivalent Linear Subgrade Static Properties

The LTR Section 5.2.1 presents, in broad terms, the approach taken to quantify:

1. The static earth pressure demands on the below-grade exterior walls are obtained using a 1g static analysis of the three-dimensional (3-D) RB FE model embedded in a layered half-space continuum model representing the surrounding soil and rock. The approach utilizes the effective weight, elastic modulus and Poisson’s ratio of the soil/rock. For a soil layer, the Poisson’s ratio is representative of the at-rest lateral pressure condition.

2. The design demands due to groundwater pressures is considered in a separate FE analysis where hydrostatic pressures are applied to the below-grade walls at elevations below the nominal groundwater level.

The LTR also states that the profiles of the equivalent linear subgrade properties for use as input to the static analysis of the BWRX-300 SMR RB are correlated with the results of nonlinear soil/rock stability analysis to ensure that the design envelopes all uncertainties related to nonlinear behavior of soil and rock mass.

The staff finds the approach to establish the static earth pressure demand on the exterior wall of the RB to be reasonable because the results from the approaches are used to establish the equivalent linear subgrade static properties. The determination of subgrade material properties is further discussed in the LTR Sections 3.2, 4.0, 5.1.2, 5.1.3, 5.2.1, and 5.2.4.

5.2.1.1 Equivalent Linear Stiffness Properties of Soil Materials

LTR Section 5.2.1.1 proposes the approach to define the stiffness properties of the subsurface materials, represented by the Young's modulus E_{st} using the effective unit weight γ and Poisson's ratio υ_{st} of the soil half-space. The LTR also proposes different correlations that can be used to convert results from field tests, such as cone penetration tests, standardized penetration tests, pressure meter tests, and dilatometer tests, in conjunction with the laboratory testing of undisturbed samples under triaxial unconsolidated undrained compression, or triaxial consolidated undrained compression to estimate the E_{st} values. The LTR, in addition, suggests the use of field measurement of shear wave velocities V_s to estimate the E_{st}. The LTR proposes the estimation of a lower bound (LB) E_{st} using methods that impose small strain in the subsurface soils. LB E_{st} values are obtained based on the weighted log-mean and log-standard deviation of the measured values using appropriate weight factors reflecting the level of confidence on the data from different field and laboratory tests.

The staff finds the approach presented is reasonable because multiple field methods and test procedures (as standardized by the American Society for Testing and Materials) are used in the different correlations which reduces the bias from the use of a single type of field or laboratory data using a statistical approach for establishing the mean and the LB estimate.

UB values for soil effective unit weight are calculated as mean plus one standard deviation of the measured values from the site investigation and laboratory tests. The staff finds this as a reasonable approach to address uncertainties in soil unit weight measurements.

The staff agrees that the value of υ_{st} can be determined from the coefficient of lateral pressure at-rest K_0, as stated in the LTR. Although LTR Section 5.2.1.1 does not specifically discuss how GEH proposes to estimate the strain-dependent modulus reduction and other dynamic properties of the soil (and also rock) following SRP Section 2.5.4, Stability of Subsurface Materials and Foundations, LTR Section 7.3, uses generic curves from Electric Power Research Institute (EPRI)-102293: "Guidelines for Determining Design Basis Ground Motions," 1993. In addition, LTR Section 3.1.2, lists several dynamic tests to estimate the strain-dependent modulus reduction and hysteretic damping properties of soil and rock (e.g., Resonant Column Torsional Shear, cyclic triaxial, Free-Free Resonant Column velocity test, etc.) at a site. The staff notes that during the review of a site-specific license application, the staff will audit these tests and approach(es) used to estimate the dynamic properties of soil (and also rock).

5.2.1.2 Rock Mass Equivalent Linear Properties

GEH has proposed several different empirical approaches using both GSI and RMR classification systems to estimate the equivalent properties of a rock mass. The GSI-based empirical approach, proposed by Hoek, E., and M.S. Diederichs, "Empirical estimation of rock mass modulus," International Journal of Rock Mechanics and Mining Science, Vol. 43, No. 2, pp 203–215, 2006, is one of the proposed approaches that use the intact rock modulus E_{ri} and the degree of rock disturbance from the excavation process D to estimate the rock mass

modulus E_{st}. D varies from 0 for undisturbed confined rock to 1 for blast damaged rock. As noted by Hoek and Diederichs, measured value of the intact rock modulus E_{ri} from undisturbed specimen is seldom available. Development of microcracks from stress relaxation can severely damage the samples. The intact rock modulus can be reduced by approximately 50 percent by these microcracks compared to undamaged samples and in situ determination of the modulus by the geophysical methods.

GEH has proposed to use the empirical equation, given by Hoek and Diederichs, to estimate intact rock modulus E_{ri} from the uniaxial compressive strength measurements using the modulus ratio values. Hoek and Diederichs give the modulus ratio, which is generally a range, for different types of rock. GEH has also proposed to use the Simplified Hoek and Diederichs empirical model, given by Hoek and Diederichs, if reliable estimation of the intact rock modulus E_{ri} cannot be made.

GEH has also proposed to use three RMR-based empirical approaches to estimate the rock mass modulus E_{st}. One of these empirical approaches is from Galera, J.M., M. Álvarez, and Z.T. Bieniawski, "Evaluation of the deformation modulus of rock masses using RMR: Comparison with dilatometer tests," Proceedings of the11th International Society of Rock Mechanics Congress, Workshop W1, Taylor & Francis, 2007, which correlates the rock mass modulus with the intact rock modulus and the RMR rating of the rock mass. GEH has also proposed to use another RMR-based approach, given Serafim, J.L. and J.P. Pereira, "Considerations of the geomechanics classification of Bieniawski," Proceedings of International Symposium Engineering Geology and Underground Construction, September. Lisbon (Portugal); 1983. p. II–33–42, for rock masses with the RMR rating of less than 50 (very poor to somewhat poor rock mass). In addition, GEH has also proposed an approach for rock masses with the RMR rating greater than or equal to 50 (fair to very good rock) by Bieniawski Z.T. "Determining rock mass deformability-experience from case histories." International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstract, 15(5): 237–47, 1978.

The staff notes that all these proposed rock mass classification-based approaches are widely used in projects around the world. The staff also notes that the approaches using the intact rock mass modulus E_{ri} in combination with the rock mass classification rating values (e.g., Hoek and Diederichs, Galera, Alvarez, and Bieniawski) are relatively recent and based on a significant years of field experience combined with measured values. As noted by Hoek and Diederichs, the Simplified Hoek and Diederichs method, as given in LTR Equation (5-20), gives larger scatter of the estimated rock mass modulus E_{st} than the equation using the intact rock modulus E_{ri}. Both RMR-based approaches, given by Serafim J.L. and J.P. Pereira, "Considerations of the geomechanics classification of Bieniawski," Proceedings of International Symposium Engineering Geology and Underground Construction, September. Lisbon (Portugal); 1983. p. II-33–42 and Bieniawski (1978), do not cover the entire range of the RMR rating values (0 to 100). In summary, based on the preceding discussion, the staff finds that the empirical approaches proposed by GEH to estimate the rock mass modulus E_{st} are reasonable; however, care should be taken for applying any of these methods to ascertain that all the conditions of its use are satisfied at the given site. In addition, the staff notes from Hoek and Diederichs that the classification schemes assume isotropic and homogeneous rock mass. This implies that a rock mass must contain a sufficient number of discontinuity sets so that its deformational behavior can be considered as isotropic. As such, a L&C # 3 has been described in Section 8.0 of this SE.

The LTR also mentions that the equivalent linear rock stiffness properties may be adjusted based on the results of the FIA model. It is not clear to the staff why these equivalent linear

rock stiffness (modulus) values need any adjustment and what would be the basis to adjust them using the FIA results. The staff will review the rationale for such adjustment in a future site-specific licensing case.

Based on the above discussion, the staff finds that GEH has proposed several alternative approaches to estimate the equivalent linear rock stiffness properties. These approaches may have some limitations based on the data sets used to develop them, as discussed above. The staff finds all these approaches are reasonable as they are used in many different mining and construction projects around the world. The staff notes that during the review of a site-specific license application, the staff will audit the estimated equivalent rock stiffness properties.

5.2.2 Development of Site-Specific Ground Motion Spectra

The LTR Section 5.2.2 states that the development of the site-specific SSE ground motion for the seismic design of SC-I SSCs should utilize a site-specific probabilistic seismic hazard analysis (PSHA) using models relevant to the site selected. Using guidance in RG 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion," Revision 0, March 2007 (ADAMS Accession No. ML070310619) and SRP Section 3.7.1, "Seismic Design Parameters," 2014 (ADAMS Accession No. ML14198A460), applicants referencing the LTR should develop a site-specific GMRS. In addition, applicants referencing the LTR should calculate the foundation input response spectra (FIRS), performance based intermediate response spectra (PBIRS), and performance based surface response spectra (PBSRS) using appropriate transfer functions developed by performing a site response analysis. The final response spectra are the result of the base rock or reference PSHA results convolved with a site-specific site response calculation performed using Approach 3 as defined in NUREG/CR–6728, "Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines," October 2001 (ADAMS Accession No. ML013100232). The LTR states that the site response calculation should be performed using two types of variability in the dynamic material properties underlying the site: (1) the aleatory variability of the site should be modeled to account for random variations in material properties within the site boundary, and (2) the epistemic uncertainty should be accounted for by multiple base-case profiles. These base-case profiles and parameters that determine the amount of aleatory variability should be determined based on the at-site geotechnical investigations.

The LTR states that in order to estimate the vertical spectrum, applicants should use the vertical/horizontal (V/H) ratios provided in NUREG/CR–6728 or in recent scientific publications on the topic. Because different spectra are calculated for different purposes (e.g., the GMRS, which defines the free-field motion at a competent surface versus the FIRS, which defines the seismic demand at the foundation level), different V/H ratios may be appropriate for different response spectra.

The LTR also states that, in accordance with 10 CFR Part 50, Appendix S, the site-specific seismic hazard results must exceed the five-percent damped spectra defined by RG 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 2, July 2014, (ADAMS Accession No. ML13210A432) anchored at 0.1g. This ensures that the site-specific aspects of the BWRX-300 SMR structures seismic design complies with the minimum SSE of 0.1g, as defined in Appendix S to 10 CFR Part 50.

The staff finds that GEH's requirements for the development of a site-specific SSE and supporting response spectra follows the current NRC guidance for meeting 10 CFR Part 50,

Appendix S and 10 CFR 100.23(d) for the determination of the site-specific seismic hazard. However, because the seismic hazard is site-specific, the staff notes that it will perform a detailed evaluation and make its final conclusions on seismic hazards during any future site-specific license review of the BWRX-300 SMR.

5.2.3 Development of Ground Motion Acceleration Time Histories

LTR Section 5.2.3 outlines an approach for developing ground motion acceleration-time histories for input in SSI analyses. The approach outlined in LTR Section 5.2.3 follows the approach described in SRP Section 3.7.1, and DC/COL–ISG–01, "Interim Staff Guidance on Seismic Issues Associated with High Frequency Ground Motion in DC and Combined License Applications," NRC, May 2008 (ADAMS Accession No. ML081400293).

The staff finds that the approach for developing the acceleration-time histories is acceptable because relevant NRC guidance and acceptance criteria in the SRP are used to determine the development and applicability of the acceleration-time histories.

5.2.4 Strain Compatible Subgrade Dynamic Properties

In LTR Section 5.2.4, GEH presents the methodology for establishing the strain-compatible dynamic soil column properties for the deterministic SSI analysis using ASCE/SEI 4-16, Section 5.1.4. To address the uncertainties related to the determination and variation of subgrade conditions, a set of three seismic SSI analyses are performed using the Best Estimate (BE), LB, and UB subgrade profiles reflecting the as-built site conditions of the site. Section 5.1.7(c) of ASCE/SEI 4-16, guidance recommends that the properties used as input to the SSI analyses be consistent with the soil properties used in the generation of input motion. The soil profiles used in the probabilistic SRA described in Section 5.2.2 of the LTR are used for development of the SSI analysis profiles of strain-compatible dynamic subgrade properties, which are consistent with the probabilistically based design motions. The effects of primary nonlinearity of the subgrade materials response are addressed by using an equivalent linear representation of dynamic stiffness and damping properties compatible to the free-field strains induced by an SSE.

The staff finds the approach to develop the soil profiles for the SSI analysis is reasonable as it maintains the consistency of the seismic motion developed in the SRA and captures the uncertainty associated with the variability of the soil properties across the site. The staff finds assurance in the fact that this practice has been used in previous license applications and has produced reasonable results.

In addition, the LTR provides a new approach to develop the Hazard Consistent Strain-Compatible Properties (HCSCP). This new approach assumes strain-compatible properties are approximately log-normally distributed, consistent with observed strong ground motion parameters (NRC, 2001) and makes use of the distributions of strain-compatible properties cataloged during development of the suites of amplification factors. The staff finds the approach to estimate the strain-compatible dynamic properties of the subgrade materials presented in this section of the LTR to be reasonable, but the staff does not have the assurance of past regulatory practice with this approach. The staff, therefore, concludes that if this first-of-a-kind approach is applied in a site-specific application using the BWRX-300 SMR structures, the staff will audit the HCSCP approach in that application. A L&C # 4 is described in Section 8.0 of this SE.

5.3 Reactor Building Seismic Soil-Structure Interaction Analysis

Section 5.3 of the LTR states that the seismic demand of the RB is computed using the SSI code SASSI (a system for analyses of soil-structure interaction) following the analytical procedure described in Section 5 of ASCE/SEI 4-16. Linear elastic SASSI analyses are performed for the set of frequencies selected in LTR Section 5.3.2. The superposition principle, applicable to linear elastic analyses, allows the SASSI stress results obtained from dynamic and static analyses of different subproblems to be combined with the results of static analyses in seismic design load combinations and used in member design. The three deterministic SSI analyses use the strain-compatible soil columns developed using the same approach and data from the SRA along with the acceleration-time histories described in Section 5.2.3 of the LTR. The effects of non-vertically propagating shear waves on the seismic response and design of RB SSCs are addressed as described in LTR Section 5.3.3.

Although the effects of ground motion incoherency are generally neglected in the design of BWRX-300 SMR, these effects may be included in the design in hard rock high frequency sites by using the coherency functions specified in Section 2.0 of DC/COL–ISG–01 or other coherency functions adequate for the site-specific conditions. In the design, ground motion incoherency effects are included after considering the comparisons of the coherent and incoherent responses and demands, and consideration of the potential variation of the coherency function with depth.

Additionally, sensitivity of the SSI analyses is considered to address the uncertainties related to the effects of excavation support and fill concrete, described in Section 5.3.8 of the LTR; the soil separation, discussed in Section 5.3.9 of the LTR; and variation of groundwater level, as discussed in Section 5.3.10 of the LTR. If these effects produce responses significantly higher (>10%) than the design-basis analyses, the results of the sensitivity analysis would be incorporated in the seismic design of the BWRX-300 SMR to bound these uncertainties.

In the LTR, GEH has proposed the SASSI extended subtraction method (ESM) simplification as an alternate approach to calculate the SSI system impedance matrix. In this method, only a selected set of nodes of the excavated volume are specified as the interaction nodes. These interaction nodes are established in the ESM model at: (1) the interfaces between the excavated volume and structural models, (2) the top surface of the excavated volume located at the PBSRS elevation, and (3) planes within the excavated volume located at PBIRS elevations. GEH proposes to use additional interaction nodes in layers of softer soil material to improve the accuracy of the SSI solution. The accuracy of the solutions obtained from the ESM analyses would be demonstrated following the guidelines provided in SRP Section 3.7.2.

GEH proposes to validate the ESM results by analyzing reduced (quarter or half)-size models. These models will be analyzed using the ESM and the SASSI flexible volume or direct method (DM) with all nodes of the excavated volume specified as interaction nodes.

If the site is located in a high seismic region and the results of the nonlinear static FIA, described in Section 4.0, indicate that the nonlinear response of the subgrade materials is significant, seismic SSI analyses will be performed using the nonlinear SSI models described in LTR Section 5.3.11 to assess the sensitivity of the RB seismic response and design on the nonlinear effects.

The staff finds the approaches to include the SSI effects in the seismic design of the BWRX-300 SMR to be reasonable. The approaches described to include the effects of ground motion

incoherency, the sensitivity of excavation materials, groundwater elevation changes, and soil separation in the SSI analysis are aligned with accepted practice. The staff finds assurance in the fact that the methods proposed are well accepted methods and have been used in the past to yield acceptable results. Use of nonlinear analysis in SSI assessment is not a common approach in licensing of the nuclear power plants; however, the staff will rely heavily on the information that is provided in a future site-specific license review of the BWRX-300 SMR to characterize and model the nonlinear behavior. A L&C # 5 has been provided in Section 8.0 of this SE.

5.3.1 Key Seismic Responses

The LTR proposes the use of key nodal locations to compare the response at these locations from the variation of the SSI parameters in the analysis. The key locations will be selected using the following criteria:

- Nodes at intersections of main structural members (main structural walls) at ground and other major floor elevations to capture global responses.
- At least two roof nodes, one central and one corner node, to show all important modes of seismic response of structure including the effects of rocking and torsion.
- At least two basement nodes, one central and one corner node, to show the SSI effects on the translational as well as the rotational (rocking and torsion) responses of the foundation.
- For the below-grade portion of the RB structure, cross-sections subjected to high seismic stress demands.

The approach of using key response locations and locations of high stress demands to compare the effect of the variation in SSI parameters is reasonable because these are important locations for assessing the effects of variation of SSI parameters on the design of the RB structure.

5.3.2 Frequencies of Analysis

The solution for the response of the SSI system will be obtained over a selected set of frequency points and then interpolated for other frequency points. The analysis is performed up to a cut-off frequency value established based on the largest value required following the four criteria of ASCE/SEI 4-16, Section 5.3.5(b). The highest dominant frequency is determined for each SSI analysis based on the acceleration transfer function representing the in-structure responses at the selected key locations, described in Section 5.3.1 of the LTR. The cut-off frequency determined by the criterion will be used in the analysis using the stiffest subgrade profile and the UB structural stiffness properties.

For lower values of the cut-off frequency, the analyses use softer subgrade profiles with reduced structural stiffness properties. In such cases, GEH commits in the LTR to demonstrate that the analysis of the stiffest profile provides responses that are bounding for frequencies higher than the cut-off frequencies used for the analyses with the softer subgrade profiles by comparing transfer function and 5 percent damped in-structure response spectra (ISRS) at the key locations selected within the structure. The frequencies of analysis are conducted at

sufficiently small frequency intervals. Transfer function amplitude calculated at the key locations will be inspected to detect any numerical anomalies in the interpolated transfer functions (e.g., sharp narrow spikes) that can potentially affect the accuracy of the results. If any numerical anomalies are present, GEH commits to evaluate the effects of these anomalies in the interpolated transfer function using analysis at additional frequencies to ensure that the anomalies in the transfer function interpolations do not affect the accuracy of the calculated responses.

The staff finds that these approaches are reasonable because they have been used in prior licensing applications and are well documented in industry consensus documents. Due to previous experience with this approach, the staff has reasonable assurance that the approach, when utilized, will yield acceptable results.

5.3.3 Effects of Non-Vertically Propagating Seismic Waves

The LTR Section 5.3.3 proposes use of sensitivity evaluations to address the effect on the SSI analysis from non-vertically propagating seismic waves for the BWRX-300 SMR design, a significant portion of which is embedded in the subsurface. Non-vertically propagating seismic waves may result in different site response and SSI results than would be expected using vertically propagating waves. No specific approach has been identified but it is suggested in the LTR that the effects of 2-D or 3-D wave propagation are to be addressed which may result from site characteristics like dipping bedrock surfaces, dipping subgrade layers, topographic effects, and other impedance boundaries, as well as the effects of local seismic sources generating inclined waves. The LTR states that if significant multi-dimensional effects are anticipated, a site-specific sensitivity analysis is performed to confirm the conclusions of the 1-D analysis.

The NRC staff finds that the approach is acceptable as it explicitly accounts for the site-specific nature of the site response analysis and the potential for multi-dimensional site effects at highly variable sites.

5.3.3.1 Evaluations of Multidimensional Wave Propagation Effects

The LTR proposes to use 1-D wave propagation analysis for rock and soil layer dipping less than the limits presented in NUREG/CR–0693, “Seismic Input and Soil Structure Interaction,” February 1979. Multidimensional, 2-D or 3-D, wave propagation sensitivity analyses may be required to study the potential generation of inclined seismic waves when site characteristics significantly deviate from the basic assumption of infinite horizontal layers. These deterministic sensitivity SRAs are typically to be performed on two models with the same subgrade material properties and configuration as the BE base-case profiles used for the 1-D SRAs described in Section 5.2.2 of the LTR. Two sets of deterministic SRAs would be performed on models representing:

1. The base-case profile used for the probabilistic SRA that assumes idealized site conditions with infinite horizontal layers, and

2. The actual site characteristics including dipping bedrock surfaces, dipping subgrade layers, topographic effects, and impedance boundaries.

Control motions may be applied to these SRA models at the bedrock surface elevations where the site reference seismic hazard is defined. The amplitude and frequency content of the input control motions are selected based on the PSHA results for rock-based Uniform Hazard

Response Spectra (UHRS) with the exceedance frequencies of 10^{-4} and 10^{-5} per year. For the sites where the nonlinearity of the subgrade materials can have a significant effect on the site response, equivalent linear sensitivity SRA would be performed using two or more UHRS controlling earthquakes with energy contents that dominate appropriate frequency ranges. For example, two control motions may be used as representative of a high frequency earthquake that dominates at high frequency range (5 and 10 Hz) and a low frequency earthquake that dominates at low frequency range (1 and 2.5 Hz). Acceleration-time histories or the Random Vibration Theory control motions may be used that match the spectral shapes generated from the reference site UHRS.

Site amplification factors are calculated based on the 5 percent damped Acceleration Response Spectra results of each deterministic SRA for the site response at the FIRS, PBSRS, and PBIRS elevations. Comparisons are made of the amplification results obtained from the SRA of model representing 1-D and multi-dimensional site conditions to determine if the site characteristics increase, decrease, or produce similar site response results. Based on these comparisons, the FIRS, PBIRS, and PBSRS, developed based on the results of 1-D probabilistic SRA analyses as described in Section 5.2.2 of the LTR, may be increased.

The staff finds this approach to addressing dipping layers in the subsurface reasonable because the effect of wave propagation due to dipping layers will be addressed in the seismic demand, when needed.

5.3.3.2 Evaluation of Local Seismic Source Effects

The presence of a local seismic source may also generate inclined waves due to the potential source-to-site effects on the wavefield. Generally, the angle of incidence of the seismic waves decreases as the waves propagate towards the ground surface due to Snell's law. Thus, non-uniform sites with softer soil layers create a vertical velocity gradient and the effects of inclined waves are reduced due to this decrease of the angle of incidence. NUREG/CR–6728 indicates rock sites at distances from the source of about 10 to 15 km or less show inclined shear wave motions. Substantial inclined shear wave motions are not shown for rock and soil sites at distances of more than 15 km from the source. Therefore, for these sites, the local seismic source effects on the BWRX-300 SMR seismic design can be neglected.

To address the inclined seismic sources that may influence the SSI analysis, the LTR proposes to use the guidance in NUREG/CR–6896, "Assessment of Seismic Analysis Methodologies for Deeply Embedded Nuclear Power Plant Structures," February 2006 (ADAMS Accession No. ML060820521) that SH waves, representing the horizontal component of the inclined shear waves, have little effect on the SSI response at the basemat level while the SV waves, representing the vertical component, influence the peak vertical response. Therefore, the LTR has not considered the effect of SH on the seismic demand for design of the BWRX-300 SMR. NUREG/CR–6896 further establishes that the effect of the SV waves is maximum when the angle of incidence is near the critical angle of incidence $\emptyset_{cr}$. GEH proposes to evaluate the effects of the inclined shear waves on the design of the BWRX-300 RB by considering SV waves at two different inclination angles $\emptyset_{cr}/2$ and $\emptyset_{cr}$ using a two-step approach. Effects of the inclined SV waves on the free-field response as well as on the FIRS, PBSRS, and PBIRS will be assessed using a free-field model without any structure in the first step. If the results indicate significant effects (>10%) of the inclined SV waves, then the SSI model of the BWRX-300 RB is evaluated with inclined SV waves.

The staff concludes that a reasonable approach has been presented in this section of the LTR for addressing the effects of potential sources of inclined waves and their impact on the design of the BWRX-300 SMR because the approach is consistent with the NRC guidance NUREG/CR–6896. The staff, in addition, takes assurance that the basis used for segregating the SH and SV waves is based on information in NUREG/CR–6896.

5.3.4 Approaches for Meeting DC/COL–ISG–017 Guidance

The intent of DC/COL–ISG–017, "Interim Staff Guidance on Ensuring Hazard Consistent Seismic Input for Site Response and Soil-Structure Interaction Analyses," March 2010 (ADAMS Accession No. ML100570203), is to ensure that the deterministic SSI analysis of embedded RB structure uses ground motion inputs that are hazard consistent with the results of probabilistic SRA, described in Section 5.2, at the foundation bottom elevation and at ground surface. For the deeply embedded BWRX-300 RB structure, the same criterion is applied to other intermediate elevations throughout the height of the embedment to provide consistency between deterministic SSI analysis and probabilistic SRA through the entire depth of the embedment. The consistency between the free-field motion for the deterministic SSI analysis and probabilistic SRA is checked at the ground surface and at intermediate elevations along the embedment depth using the PBSRS and PBIRS developed, as described in Section 5.2.2 of the LTR. The elevations corresponding to significant V_S contrasts in the SSI soil profiles are included as intermediate elevations for the checks.

The LTR proposes any of the following three approaches for the consistency checks:

1. Perform the checks prescribed in Section 3.2.3, "SSI Analysis of Embedded Structures Including Embedment," in Nuclear Energy Institute, (NEI) White Paper, "Consistent Site-Response/Soil-Structure Interaction Analysis and Evaluation," June 2009 (ADAMS Accession Number ML091680715), (NEI, WP 2009) and discussed in LTR Section 5.3.4.1 to ensure that the horizontal and vertical FIRS applied to the model at the bottom of the RB foundation is adequate at the ground surface and throughout the embedment depth.

2. Envelop the results of three or more sets of SSI analyses, described in Section 5.3.4.2 of the LTR, performed with FIRS, PBSRS, and PBIRS defined input ground motions applied at the foundation bottom, ground surface, and intermediate elevations, respectively.

3. Perform the checks, as described in Section 5.3.4.3 of the LTR, only for the horizontal direction and using the vertical free-field input motion for the SSI analysis that is constrained along the embedment depth of the soil columns based on the V/H ratios used for the probabilistic SRA and following the methodology in EPRI Report 3002011804, "Advanced Nuclear Technology: Modeling Vertical Free-Field Motion for Soil-Structure Interaction of Embedded Structures," 2018, described in Section 5.3.4.3.

As an alternative, the LTR proposes to conduct a probabilistic SSI analysis following the requirements of Section 5.5 of ASCE/SEI 4-16 that would satisfy the DC/COL–ISG–017.

The staff finds all the proposed approaches reasonable and suitable for demonstrating that a hazard consistent spectrum has been used in the design of the deeply embedded BWRX-300 RB structure and they are evaluated below.

5.3.4.1 NEI Checks of FIRS Defined Input Ground Motion

The staff has reviewed the information GEH has submitted in LTR Section 5.3.4.1. GEH would check the input motion to be applied at the bottom of the foundation of the structures following the procedure of NEI WP 2009. These checks would be conducted for both horizontal and vertical component of the ground motion by performing 1-D linear elastic SRA on the same set of strain-compatible compression wave velocity V_p and shear wave V_s velocity profiles used in the deterministic SSI analysis. For all V_s profiles, the free-field motions calculated in the horizontal direction at the ground surface and at selected intermediate elevations of the deeply embedded reactor are enveloped. Similarly, the motions in the vertical direction for all V_p profiles are enveloped. This enveloped motion at the ground surface would be compared with the horizontal and vertical PBSRS. The enveloped motion at all the intermediate elevations would be compared with the PBIRS.

The staff finds the approach described in this section of the LTR to be reasonable because it is consistent with the NEI WP 2009. However, the verification of the NEI checks will be a critical part of the staff review of a site-specific future license application of the BWRX-300 SMR.

5.3.4.2 FIRS and PBSRS Defined Input Ground Motions

In this approach, GEH proposes to envelop the results from multiple sets of SSI analyses using the BE, LB, and UB subgrade profiles with input motion compatible to:

1. The FIRS and applied at the bottom of the RB foundation in the SSI model.

2. The PBSRS and applied at the ground surface in the SSI model.

3. The PBIRS calculated at selected intermediate locations and applied at corresponding locations of the SSI model.

This proposed approach ensures that the free-field probabilistic site response analysis is enveloped by the design.

The staff finds this approach reasonable for demonstrating that a hazard consistent spectrum has been used for design of the BWRX-300 SMR as it is consistent with the guidance of DC/COL–ISG–017.

5.3.4.3 V/H Based Vertical SSI Input Motion

In this approach, the checks, as described in Section 3.2.3 of the NEI WP 2009, are conducted only in the horizontal direction. As discussed in EPRI 2018, the vertical motion applied at the foundation level amplifies as it reaches the ground surface and the V/H ratio exceeding the value determined at the site. This overestimation of the vertical design ground motion results in an overly conservative design of the structure and the equipment. GEH proposes to follow the method described in EPRI 2018. The site V/H ratio is used to develop the free-field ground motion for SSI analysis. GEH will check the accuracy of the vertical motion applied to the SSI model along the embedment depth at the free-field interaction nodes. The resulting V/H ratios are compared with the V/H ratios used to generate the vertical PBSRS and PBIRS.

The staff finds the proposed approach for demonstrating that a hazard consistent spectrum has been used to design the deeply embedded BWRX-300 RB structure to be reasonable and suitable because it uses the method suggested by EPRI 2018.

5.3.5 Effects of Variation of Structural Stiffness and Damping Properties

The modeling of appropriate stiffness and damping properties of the structural members in the SSI model is essential for the accuracy of the calculated seismic responses and seismic demands. The stiffness of concrete structural members, such as reinforced concrete or steel-plate composite (SC) members, depend on the degree of concrete cracking. Effects of concrete cracking on structural stiffness is considered by the following approaches.

Stiffnesses of the reinforced concrete members, calculated per ACI 349-13, are reduced based on the criteria provided in Table 3-1 of ASCE/SEI 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,” 2005, to address the effects of cracking. The cracking status of a reinforced concrete member is evaluated based on the recommendations in Section 3.3.2 of ASCE/SEI 4-16, using the nominal concrete compressive strength (f_c') and the overall level of stress the structural member experiences under the earthquake design loads in combination with other applicable design loads. ASCE 43-19, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,” 2019 is the latest update of the ASCE/SEI 43-05 standard. The effective stiffness of SC walls is determined based on Section N9.2.2 of AISC N690-18. The effective in-plane shear stiffness of SC walls is determined from the equations provided in Section N9.2.2(b) of AISC N690-18.

In accordance with SRP Section 3.7.2 and following the guidance and the requirements of Section 3.3.2 of ASCE 4-16, GEH states that the analyses will be performed on models that represent the uncracked stiffness properties of the concrete. Depending on the level of stress in the concrete due to the most critical seismic load combinations, effective stiffness values are assigned to the concrete members depending on their cracking status. The assignment of the stiffness properties to structural members follows the guidance in SRP Section 3.7.2.

GEH has proposed to use an optional approach, presented in Section C3.3.2 of ASCE/SEI 4-16, to design the BWRX-300 SMR. This approach uses the design-basis model with stiffness properties that yield conservative seismic responses and design for the site-specific conditions. This approach can also be used to address the effects of variations of structural stiffnesses. A third approach suggested in the LTR is to capture the structural stiffness variation using a sensitivity SSI analysis for the following bounding stiffness conditions:

a. A fully cracked condition when all concrete structural members are fully cracked and are assigned higher SSE damping properties; and

b. A fully uncracked condition when all concrete structural members are assigned full (uncracked concrete) stiffness and lower Operating Basis Earthquake (OBE) damping properties.

These sensitivity analyses are performed for the BE subgrade profile to evaluate possible amplifications of in-structure responses and load demands on the steel members from the load redistribution effects. These evaluations are based on comparisons of results from these two sensitivity analyses and the design-basis analysis performed for the BE profile using the BE dynamic properties for the RB structure. The comparisons are performed for in-structural

responses and stress demands at key locations selected, as described in Section 5.3.1 of the LTR.

The damping ratio assigned to the structural members should be consistent with the cracked or uncracked state or as an alternate they can be assigned in accordance with Section C.1.2 of RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," Revision 1, March 2007 (ADAMS Accession No. ML070260029). The uncracked members in the models used for calculation of the ISRS and other in-structure response demands for seismic design and evaluation of seismic Category I (SC-I) equipment and components, are assigned lower OBE damping values.

The staff finds this approach to capture the variation of the stiffness of the structural model under the seismic event is reasonable because these methods are well established practices and captured in the industry consensus documents and RGs.

5.3.6 Dynamic Modeling of Subsystems, Components, and Equipment

In this section of the LTR, GEH proposes to include subsystems, components, and equipment (referring them collectively as equipment in subsequent discussion) in the SSI analysis model based on their mass ratios and first natural frequencies. The RPV will be modeled as a lumped-mass stick model capturing all significant seismic response. The equipment-structure interaction (ESI) effects would be explicitly considered in the analysis by using either of the three methods: (1) DM (explicitly modeling the equipment mass and stiffness), (2) Mass-Impedance ESI Method (mass of the equipment and dynamic stiffness represented as impedance), or (3) Generalized ESI Method (consideration of a secondary system with multiple degrees of freedom attached to the structure at multiple points). Modification of the ISRS due to ESI would be calculated.

The staff finds the proposed approaches reasonable because they are typically used in the industry. Whether a particular equipment should be included in the SSI analysis would be determined following the criteria described in Subsection II.3.B of SRP Section 3.7.2. The proposed three methods to analyze the ESI effects are also used in practice as outlined in EPRI Report 3002009429: "Advanced Nuclear Technology: High-Frequency Seismic Loading Evaluation for Standard Nuclear Power Plants," 2017.

5.3.7 Modeling of Structure-Soil-Structure Interaction Effects

In LTR Section 5.3.7, GEH presents the approach to address the Structure-Soil-Structure interaction (SSSI) effects that result from the proximity of the RwB, CB, and TB on the SSI of the RB. The increased overburden from the buildings can have significant effect on the lateral loads applied to the RB below-grade walls and to some extent impact the RB SSI effect.

Simple models representing the BE dynamic properties of surrounding buildings and foundations will be included in the RB FE model used for the seismic SSI analysis. These simple models are sufficiently refined to capture all global modes of vibration of RwB, CB, and TB structures with significant (> 20%) modal mass participations in the three orthogonal directions.

The staff finds this approach to include the SSSI effects in the SSI analysis of the BWRX-300 RB structure reasonable because similar approaches have been used in prior licensing applications to include the effects of SSSI.

5.3.8 Excavation Support and Backfill Effects

The LTR states that the preferred method of construction of the BWRX-300 RB shaft is the open caisson method. The excavation for the shaft in softer soil strata is retained by a circular slurry shoring wall socketed into bedrock. The concrete structure for the RB is constructed bottom up within the caisson.

In LTR Section 5.3.8, GEH states that the BWRX-300 SMR design does not rely on the resistance provided by the slurry wall or other support systems used to secure the stability of excavation and the lean concrete used to fill the gap between the below-grade RB shaft exterior wall and the excavated soil and rock. These construction elements are temporary by design and are excluded from the models used for the static and dynamic SSI analysis because they are not expected to maintain their structural integrity through the entire operational life of the plant. The exclusion of the excavation supports and fill concrete results in conservative estimates of the static and dynamic lateral pressure demands on the RB below-grade walls.

The staff concludes that with the degradation of the excavation slurry wall the lateral soil pressures on the RB exterior wall computed based on K_0 would be smaller and, hence, using K_0 would provide a conservative estimate for the design demand.

The LTR also discusses the potential effects of the RB shaft construction and waterproofing on the friction at the interfaces between the exterior walls of the RB and the surrounding excavation support structure or the fill materials that may be used. GEH has proposed to conduct sensitivity analyses to address the uncertainties of the friction at the RB shaft interfaces by conducting analyses at two extreme frictional conditions: (1) fully bonded with no slippage, and (2) no frictional resistance. If the calculated ISRS and force demands at selected key locations (LTR Section 5.3.1) significantly (> 10%) differ from the results of design-basis SSI analyses, the results of these sensitivity analyses would be included in the RB seismic design-basis. The staff finds the proposed approach to be reasonable as it bounds the effects of friction from RB shaft construction and waterproofing and to assess their effects on the design-basis SSI analyses. In addition, a path forward is proposed if the effects are significant.

5.3.9 Soil Separation Effects

GEH has proposed in this section of the LTR to assess the effects of soil separation using the guidance in ASCE/SEI 4-16, Section 5.1.9(b) by comparing the difference between the seismic and the static lateral earth pressure on the wall of the RB shaft, calculated from the 1g static SSI analysis, in lieu of a nonlinear SSI analysis. The regions where the static lateral pressure $\rho_{LB}(z)$ is lower than the seismic lateral pressure calculated from the seismic SSI analysis indicate potential separation at the soil-structure interface.

The staff finds this approach to establish the non-contact surface over the height of the RB wall and the excavated face to be reasonable because the guidance in ASCE/SEI 4-16 Section 5.1.9(b) is a widely accepted consensus approach which consistently demonstrates acceptable results.

5.3.10 Groundwater Variation Effects

Variations in the groundwater level can change the dynamic properties of the subsurface soil/rock and affect the seismic response of the RB and the in-structure responses. GEH has

proposed to address this issue by performing a sensitivity analysis using two extreme water level conditions: (1) a fully wet soil profile (simulating a flooded site) and (2) a dry soil profile (when ground water is assumed to be below the foundation level). Both analyses will be conducted using the BE dynamic soil properties. If the results vary greater than 10 percent at the key locations, the design-basis would be developed based on a fully saturated soil profile below the nominal groundwater table.

The staff finds this approach to addressing the effects of ground water table variation on the result of the seismic response of SSC's reasonable because the two extreme conditions considered in the analysis fully bound the effects of water table fluctuations on the seismic design of the RB.

5.3.11 Non-Linear Seismic SSI Analysis

If the site selected for constructing a BWRX-300 SMR is in a high seismic region and/or the subgrade materials exhibit highly nonlinear behavior, GEH proposes to use the nonlinear SSI analyses, following the guidance given in Appendix B of ASCE/SEI 4-16, to assess the importance of the following on the RB seismic response and design: (1) the secondary nonlinearity of the subgrade materials including nonlinearities introduced by the slip and opening of the rock discontinuities and (2) any nonlinearities introduced by separation and sliding of the soil/rock-structure interfaces. Although there is some evidence that plastic deformation of the subgrade materials reduces the structural response, especially at high frequencies, GEH proposes to analyze any potential amplification of the RB structural response from the secondary nonlinearity of the subgrade materials. The proposed analyses would particularly assess whether presence of fracture zones, rock joints, bedding planes, discontinuities, cavities, and other weak zones in the rock mass may significantly amplify the rock pressure loads and affect the block stability. These analyses will use nonlinear constitutive models with the BE properties of the subgrade materials, as recommended in Section B.4 of ASCE/SEI 4-16.

If the separation at the soil/rock-structure interface is found to be significant, as discussed in LTR Section 5.3.9, SSI analyses would be performed to explicitly assess the potential nonlinearities at these interfaces. Because the focus of these analyses is to assess the effects of the nonlinear behavior of the subgrade materials, the structural members would be assigned linear elastic properties with the BE stiffness and damping properties. GEH states that they will eliminate any unintended numerical damping introduced by the numerical integration in the SSI model. Additionally, the model boundaries should be placed at an adequate distance away to simulate the semi-infinite boundary conditions of the subgrade. The Domain Reduction Method may be used to analyze such scenarios to reduce computational resources. Following Section B.3 of ASCE/SEI 4-16, three components of the ground motion would be simultaneously applied to the SSI model. Results of the nonlinear SSI analyses would be compared with the linear elastic SSI analyses to assess the effects of these nonlinear phenomena at the subgrade in addition to the ISRS and member forces calculated. If the nonlinear effects are significantly more than 10 percent, GEH would adjust the seismic design of the RB structure to envelope the nonlinear effects.

The staff finds the intent to conduct nonlinear SSI analyses to capture the effects of any significant nonlinear response of the subgrade materials on the RB seismic response and design to be reasonable. The analyses would be conducted following the guidance given in Appendix B of ASCE/SEI 4-16, a nationally recognized standard. The proposal to compare the nonlinear SSI analysis results with those from the linear SSI analysis is also reasonable. As

stated in SRP Section 3.7.2, the staff conducts a detailed review of all inelastic/nonlinear analyses. A L&C # 5 has been provided in Section 8.0 of this SE.

6.0 DESIGN APPROACH FOR II/I INTERACTION

GEH presents in this section of the LTR a graded approach with associated acceptance criteria for design and evaluations of the II/I interactions of the CB, TB, and RwB structures with the deeply embedded RB structure. CB, TB, and RwB structures are designed according to their seismic classification. Design of these structures would be evaluated for SSE and the design-basis tornadoes, hurricanes, and extreme wind loads to assess whether they meet II/I interaction guidance, as given in Subsection II.8 of SRP Section 3.7.2 and listed in Section 2.4 of this SE.

The evaluation should also demonstrate that no gross failure occurs in the CB, RwB, and TB structures. Additionally, the structural displacement from these events would be accommodated by the gap provided among these structures and the RB structure. Definitions of the design-basis tornadoes and hurricanes are from RG 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," Revision 1, March 2007 (ADAMS Accession No. ML070360253) and RG 1.221, "Design-Basis Hurricane and Hurricane Missiles for Nuclear Power Plants," Revision 0, October 2011 (ADAMS Accession No. ML110940300), respectively.

GEH proposes to conduct evaluations of the II/I interactions with limited inelastic deformations following guidance in Subsection II.8 of SRP Section 3.7.2. To satisfy Criterion C of Subsection II.8 of SRP Section 3.7.2, the gap would be considered adequate if it is larger than the absolute sum of displacements of each structure along the entire height considering construction tolerances. GEH states that gross failure of these structures would be prevented as they would be designed following the applicable design codes and standards.

The staff finds the description presented in this section of the proposed evaluations of the II/I interaction between the RB structure and the CB, TB, and RwB structures reasonable because the proposed evaluations are based on the guidance given in Subsection II.8 of SRP Section 3.7.2. In addition, construction tolerances would be included in determining the gaps among these structures, consistent with SRP Section 3.7.2, making the gap assessment robust.

6.1 Control Building, Turbine Building, and Radwaste Building Design Bases

In this section, GEH states that the CB and TB structures are considered non-seismic and the RwB structure is considered RW-IIa category, as specified in RG 1.143, "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants," Revision 2, November 2001 (ADAMS Accession No. ML013100305), Section 5.1. These structures would be designed based on their seismic classification. The staff finds this classification of these structures reasonable as further evaluated below.

6.1.1 Non-Seismic Control Building and Turbine Building Structures and Foundations Design Bases

The staff has reviewed the discussion given in this section of the LTR regarding design and construction of the non-seismic CB and TB structures and their foundations. GEH proposes to design these non-seismic structures following Chapter 16, Structural Design, of International Code Council, "2018 IBC Code and Commentary," 2018 and applicable provisions of

ASCE/SEI 7-16. Structural concrete will be designed in accordance with the requirements of Chapter 19 of IBC and ACI 318-14, "Building Code Requirements for Structural Concrete and Commentary," 2014. The control room may be designed as a reinforced concrete structure within the steel-framed structure of the CB. The steel framed structure of the CB will be designed, fabricated, and constructed following Chapter 22 of IBC and AISC 360-16, "Specification for Structural Steel Buildings," 2016. Both the CB and non-seismic portion of the TB would be designed as Risk Category IV structures. Design earthquake loads would be following Section 1613 of IBC and ASCE/SEI 7-16.

The staff finds that the use of building codes and national standards, such as ASCE/SEI 7-16, AISC 360-16, and ACI 318-14, to design, fabricate, and construct the non-seismic CB and TB structures to be reasonable. Use of Risk Category IV, defined in ASCE/SEI 7-16, for these structures is also appropriate because failure of these structures could pose a substantial hazard to the community.

6.1.2 Radwaste Category IIa Building Structure and Foundations Design Basis

The design guidance for radwaste structures provided in RG 1.143, is to, in part, address aspects of GDC 60 and 61 related to controlling the release of radioactive material and provides appropriate containment and confinement of radioactive material. RG 1.143 indicates that radwaste structures should be classified based on the potential radiological consequences of an unmitigated release to the public or unmitigated exposure to workers. The RW-IIa classification is the most robust design-specified in RG 1.143 for radwaste structures and RG 1.143 includes no upper limit on the unmitigated release or unmitigated exposure to workers from material in an RW-IIa structure. As a result, the staff finds that it is acceptable to classify the RwB structure as RW-IIa for the purposes of meeting the design guidance of RG 1.143.

LTR Section 1.3 stated that the portions of the TB structure and foundation that support and enclose the main steam piping and the Off-Gas System (OGS) for management of radiological gases are designed as RW-IIa following the provisions of RG 1.143. LTR Section 6.1.2, further stated that based on RG 1.143, Table 2, the loads for the design RW-IIa RwB and TB structures include one-half of the SSE seismic load. Because RG 1.143 only addresses Radioactive Waste Management Systems for which it provides guidance for the design, construction, installation, and testing the SSCs of radioactive waste management facilities in LWR nuclear power plants, GEH, in response to a staff RAI 01.05-1 dated August 19, 2021, indicated that it would move the OGS charcoal absorbers to the RwB, thereby eliminating the need for the associated portion of the TB to comply with RG 1.143. The response also stated that SSCs in the TB would not be relied on for accident mitigation and indicated that revisions would be made to Sections 1.3, 2.4, 6.1, and 6.4 of the LTR to reflect the relocation of the OGS absorbers. The staff reviewed Revision 1 of the LTR and confirmed the indicated revisions were made.

While the applicant discusses design aspects of the main steam system in the response to RAI 01.05-1, the LTR does not provide detailed design information for plant systems and does not request approval of the design of the main steam system and its associated SSCs. Therefore, the NRC staff will conduct the review of the system during future licensing activities when detailed design information for the system is submitted.

In addition, the staff notes that if following the guidance in RG 1.143, the radioactive waste management systems and components would also be given a radwaste classification based on the building classification and the quantities of radioactive material in the systems and components. However, the radwaste system design and the classification of radwaste systems

are not addressed in the LTR. Therefore, the NRC staff will conduct the review of the systems during future licensing activities when detailed design information for the system is submitted.

Likewise, while the LTR discusses design aspects of the BWRX-300 structures, the LTR does not address any aspects of the radiation protection design (e.g., radiation shielding), other than the RW-IIa RwB classification. Therefore, the staff will conduct its reviews of these aspects during future licensing activities when detailed design information is submitted.

6.2 II/I Seismic Interaction Evaluations

GEH proposes to evaluate the CB, TB, and RwB structures for II/I interactions during an SSE event to ensure:

i. Integrity of the lateral load resisting members is not compromised.

ii. Stability of the foundations of these structures are not compromised in an SSE event.

iii. Gaps between the RB and these structures are adequate to prevent any physical interactions.

II/I seismic interactions would be evaluated using the calculated responses of these structures in an SSI analysis using linear material properties. Limited inelastic responses would be considered (Limit State C, as defined in ASCE/SEI 43-05. As an alternative, GEH has proposed to conduct the Fixed-Base analysis if any of three criteria in Section 5.1.1 of ASCE/SEI 4-16 is satisfied. The SSE demands, determined using the linear elastic seismic response analysis of the CB, TB, and RwB structures, would be reduced by an appropriate reduction factor given in Table 5-1 of ASCE/SEI 43-05. Sliding and overturning stability of the foundations would be evaluated using the results of seismic analyses. The gaps between the RB and other structures would be considered adequate if they are larger than the absolute sum of the SSE-driven displacement of each structure and evaluated along the entire height of the structure. In addition, construction tolerance would be included in the evaluation. The seismic displacements calculated from linear elastic seismic models would be converted to the inelastic displacements using a factor given in ASCE/SEI 41-17, “Seismic Evaluation and Retrofit of Existing Buildings,” 2017.

The staff has reviewed the discussion on the approaches proposed to evaluate the II/I interactions between the structures in a seismic event. The staff finds that the approaches to be reasonable because the approaches appropriately use guidance given in national standards in this assessment to satisfy Criterion C of Subsection II.8 of SRP Section 3.7.2, for assessment of seismic gap from an SSE event.

6.3 II/I Interaction Evaluations for Extreme Wind Loads

In this section of the LTR, GEH discusses evaluations of II/I interactions for extreme wind events. GEH states that interaction checks would be performed in accordance with the design codes and standards using the same analytical model without the SSI. RG 1.76 gives the appropriate wind speeds for tornado loads. RG 1.221 gives the wind speeds for hurricanes appropriate for a nuclear facility at a given location. Chapter 26 of ASCE/SEI 7-16 gives the design-basis straight-line wind speeds. The CB and TB are steel-braced structures designed in accordance with ANSI/AISC 360-16, “Specification for Structural Steel Buildings, American Institute of Steel Construction,” 2016. Limited inelastic response of these steel-braced frames

would be permitted if the global stability is assured to check for interaction. ACI 349.3R (2010) requires structures to remain elastic in analyzed loadings. Therefore, the check for II/I interactions for RwB would be performed under controlling wind loads which maintain a linear elastic response. The RB structure is also designed against design-basis tornado missiles, given in RG 1.76, and hurricane missiles, given in RG 1.221. The RwB is also designed for tornado missile strikes.

The staff finds that the discussion given in checking for II/I interactions for a tornado, a hurricane, or an extreme wind event is reasonable because it uses national standards to check for any interactions among the structures in these events.

7.0 BWRX-300 SMR GENERIC DESIGN APPROACH

This section of the LTR describes the methodology to develop generic seismological and geotechnical site parameters for a wide range of conditions at candidate sites across North America. A detailed review of each section is provided below.

7.1 BWRX-300 SMR Structural Conceptual Design Approach

In this section of the LTR, GEH states that the BWRX-300 SMR is conceptually designed to use a lower amount of construction materials. GEH is also performing design calculations for sites with a range of geotechnical and seismological conditions representing at least 80 percent of all North American candidate sites. The majority of the safety-related components and equipment would be placed below-grade to mitigate potential effects of external hazards, such as adverse weather or aircraft crashes.

GEH also states that they have conducted seismic SSI response analysis using a FE model of the RB structure using eleven different generic soil profiles listed in LTR Table 7-1. GEH concludes that application of the generic conceptual design at these eleven different site conditions ensures economic viability of the BWRX-300 SMR. However, because the SSI results are not part of this LTR, the NRC staff did not evaluate the results.

7.2 BWRX-300 SMR Generic Design Response Spectra

The LTR Section 7.2 provides three generic design response spectra (GDRS) that are developed to be representative of a broad variety of regions across the United States. These three GDRS are all anchored at 0.3g for peak ground acceleration, which may not be representative of high-hazard sites. The LTR provides three response spectra for both horizontal and vertical components.

The NRC staff reviewed the GDRS and finds them to be consistent with the expected response spectra for regions across the United States, except for high-hazard sites. The GDRS are expected to meet the requirements of GDC 2 for low to moderate hazard sites, which comprise the majority of potential reactor sites within the United States.

7.3 Generic Profiles of Dynamic Subgrade Properties

The LTR Section 7.3 provides eight generic profiles for subsurface dynamic properties that are developed to be representative of a broad variety of candidate sites (LTR Figures 7-2 through 7-5). The profiles that are characterized here by the shear wave velocity range from low velocity sites (consistent with soil sites) to high velocity sites (representative of hard rock sites). The

profiles were developed by grouping measurements from multiple sites with similar subsurface properties and geologies and averaging the results. The resulting profiles are broadly representative of a number of subsurface conditions and results from these generic profiles can inform seismic design in the generic BWRX-300 SMR structures.

The NRC staff has reviewed the generic subsurface profiles and finds them to be broadly consistent with conditions found across the U.S. However, the NRC staff notes that variation of these soil properties with depth are site-dependent. Additionally, it is not clear if these profiles specifically account for presence of a rock mass in the subsurface. Therefore, the staff cannot determine whether the profiles, as given in LTR Figures 7-2 through 7-5, would be appropriate for a site in the U.S. The staff notes that these variations of the dynamic properties in the subgrade of a site would need to be provided and evaluated by the staff during the review of any future site-specific licensing application.

7.4 BWRX-300 SMR Generic Design Soil Parameters

In this section, GEH discusses six soil engineering parameters to characterize different subgrade materials for the generic conceptual design of the BWRX-300 RB structure: (1) dry and total unit weights (W_s), (2) void ratio (e), (3) internal frictional angle $(\emptyset_s)$, (4) at-rest lateral pressure coefficient (K_0), (5) active lateral pressure coefficient (K_a), and (6) passive lateral pressure coefficient $\left(K_p\right)$. GEH has provided in LTR Table 7-2 the generic soil parameters given in the design manuals by the Iowa Department of Transportation. GEH has claimed that the properties of cohesionless soils given in Table 7-2 would adequately represent the generic candidate sites. This section also discusses how some of the properties are determined from other properties given in Table 7-2.

The NRC staff has reviewed the generic design soil properties and found them to be reasonable. However, it is not clear to the staff what the basis would be to claim that the generic design soil properties would represent at least 80 percent of candidate sites in North America. The staff notes that other parameters of the subsurface soil layers would be necessary to design the BWRX-300 RB structure at a specific site. Based on the preceding discussion, the staff cannot determine whether the generic design soil parameters would represent a site as they are site-specific parameters. The staff notes that the design soil parameters of the selected site would need to be provided and evaluated during the review of any future site-specific licensing application.

7.5 BWRX-300 SMR Generic Profiles of Static Subgrade Properties

In this section, GEH has presented eight generic profiles of variation of the unit weight, Young's modulus, and Poisson's ratio with depth, as shown in LTR Figures 7-6 through 7-8. These static properties are used to determine soil pressure demand. The approach to determine different parameter values has been discussed in this section.

The NRC staff has reviewed the generic profiles of these three subgrade material properties with depth and notes that profiles of dry unit weight, soil Young's modulus, and soil Poisson's ratio with depth, as given in LTR Figures 7-6 through 7-8, are dependent on the site. The staff cannot determine whether the variations in these profiles with depth would be appropriate without a site-specific basis, and specific details of the subsurface properties. As such, the staff notes that the subgrade properties of the selected site would need to be provided and evaluated during the review of any future site-specific licensing application.

7.6 <u>BWRX-300 SMR Generic Design Base Shear Friction Coefficients</u>

GEH has provided the generic values of the friction coefficient between the concrete basemats and different types of subgrade materials underlying the basemat, as given in LTR Table 7-2. GEH has not provided the source for these assumed values but described them as common engineering practice. The staff notes that the friction coefficient is a site-specific parameter to be measured from samples taken during site investigation.

The NRC staff has reviewed the friction coefficient of the reactor base and the medium lying immediately underneath it. The NRC staff notes that these assumed values pertain to the horizontal sliding surfaces at the bottom of the RB, RwB, TB, and CB foundations and are generic. GEH will measure the friction coefficient of the vertical wall between the RB shaft and the surrounding soil/rock media in a site-specific application as one of the interface (Figure 4-2) parameters, as discussed in LTR Section 3.1.2 and response to RAI 02.05.04-01 dated November 4, 2021. The staff notes that the friction coefficient is a site-specific parameter that would need to be measured, provided, and evaluated by the staff during the review of any future site-specific licensing application.

7.7 <u>BWRX-300 SMR Generic Design Nominal Ground Water Level</u>

GEH has used ground water pressure loads assuming two ground water elevations at the site in the generic BWRX-300 design:

I. At plant grade.
II. Below the RB foundation.

The same ground water level has been used in stability calculations to account for the buoyancy force. The staff notes that this load is due to ground water in the soil/rock matrix. At some sites, flow of ground water through a rock fracture at substantial pressure may also introduce additional load (response to NRC RAI Question 02.05.04-07). Whether flow of water through fracture(s) is significant is a site-specific condition. The staff notes that the presence of significant fracture flow at the selected site would need to be provided and evaluated by the staff during the review of any future licensing application.

8.0 LIMITATIONS AND CONDITIONS

If an applicant chooses to incorporate by reference the approaches, methodologies, and laboratory and field tests to be conducted at a site, or other discussions given in this LTR as part of a DC application, or if a license applicant uses it for requesting a construction permit and operating license under 10 CFR Part 50 or a combined license under 10 CFR Part 52, it must provide appropriate safety analyses to demonstrate compliance with the applicable regulatory requirements.

In addition, the applicant referencing the LTR for construction and design features as part of a license application for approval of a reactor design, construction, and/or operating license must address in their applications the following L&Cs or provide additional justification for any deviations.

8.1 L&C # 1 Interface Characteristics Testing

As discussed in SE Section 3.1.2, large size samples collected at a site should be tested in the laboratory to have an acceptable estimate of the measured discontinuity (e.g., rock-rock, rock-soil) and interface (e.g., rock/soil-structure) strength and deformation parameters for a nuclear power plant. The NRC staff will review the sizes of the samples and their testing at the laboratory to estimate the properties of the discontinuities and interfaces in a site-specific license application with a BWRX-300 SMR.

8.2 L&C # 2 Stable Excavation

As discussed in SE Section 5.1.2, a stable shaft excavation would have no unstable blocks in its surrounding that may slide into the excavation. A self-supported (even with some temporary reinforcement) excavation would be needed to place the RB and to estimate the earth pressure loads to be considered in the generic design of the RB structure. The NRC staff review of a site-specific application with the BWRX-300 SMR will focus on the method(s) used to identify the unstable rock blocks in the area surrounding the RB shaft and to assess the earth pressure imparted on the RB shaft for determining whether the subgrade is acceptable for siting the reactor. In addition, any temporary reinforcement or mitigation measures used to stabilize the surrounding materials would be reviewed by the staff.

8.3 L&C # 3 Isotropic and Homogeneous Rock Mass

As discussed in SE Section 5.2.1.2, the rock mass classification systems inherently assume isotropic and homogeneous rock mass. This assumption therefore implies that a jointed (or a fractured) rock mass contains a sufficient number of discontinuity sets so that its deformational behavior may be assumed to be isotropic and homogeneous. The NRC staff will review whether the discontinuity sets at the selected site would make the rock mass behavior isotropic and homogeneous in any future site-specific licensing application.

8.4 L&C # 4 Site Specific Application of the HCSCP

In Section 5.2.4 of the LTR, GEH proposed a new approach to develop the HCSCP. Although the approach is reasonable, it will be the first ever application to a nuclear reactor project. The staff notes that during the review of future licensing applications, the staff will audit the HCSCP approach.

8.5 L&C # 5 Nonlinear SSI Analysis

GEH proposed in Sections 5.3 and 5.3.11 of the LTR to conduct a nonlinear sensitivity SSI analysis, as necessary, to validate any nonlinear effects from high seismicity and/or subgrade materials on the RB seismic response and design. The NRC staff plans to review the characterization and modeling of the nonlinear behavior of the materials surrounding the reactor in any future licensing application utilizing a nonlinear SSI analysis approach.

9.0 CONCLUSION

Based on the above discussion, the NRC staff concludes that the proposed construction and design approaches for the BWRX-300 SMR, as described in this LTR, are acceptable with some limitations and conditions. In particular, the LTR describes approaches to address the design analysis and construction of a BWRX-300 nuclear plant with a proposed below-grade RB shaft. Also, as discussed in this SE, GEH has indicated that the detailed design of the BWRX-300 SMR has not been complete at this time. As such, until the detailed design is completed, or the identified site-specific aspects are identified, five L&Cs for the use of this report are identified and summarized in Section 8 of this SE.

Additionally, as stated above, upon implementation of this LTR into a site-specific application of the BWRX-300 design, the staff will evaluate each topical area designated above to ensure that each topic appropriately interfaces with the proposed license application to ensure consistency. The staff will also make its regulatory determinations regarding the topics discussed above, as applicable, during its review of any future license application.

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REVISION SUMMARY

Revision Number	Description of Change
0	Initial Revision

1	Revised to incorporate the following responses to NRC Requests for Additional Information (eRAIs):
	• NRC eRAI 9849 Question 01.05-01 revised Sections 1.3, 2.4, 6.1, 6.1.2, and 6.4 to reflect the relocation of the offgas system charcoal adsorbers to the Radwaste Building.
	• NRC eRAI 9859 Question 02.05.04-01 revised the following sections:
	  o Section 3.1.2 to include direct shear and triaxial strength tests for natural and artificial discontinuities.
	  o Section 3.1.3 to identify direct shear and triaxial compressive tests as laboratory tests on recovered samples of discontinuities.
	  o Section 4.3.1.1 to state that the interface parameters are from adjacent soil/rock elements or strength test on natural and artificial discontinuities developed to be consistent with the selected nonlinear FIA software and interface model.
	  o Section 4.3.1.2 to indicate that the weakest strength parameters from multiple tests on rock discontinuities may be used for interface elements.
	  o Section 8.0 to add references for ASTM D5607, RTH 203-80, RTH 204-80, RG 1.132 and RG 1.138.
	• NRC eRAI 9859 Question 02.05.04-02 revised the following sections:
	  o Section 3.1.1 to identify the need for geologic mapping of outcrops to improve the field investigation program and the rock characterization when bedrock is encountered at depths for engineering purposes.
	  o Table 3-1 to add “characterize rock mass and discontinuities”.
	  o Section 3.1.3 to identify the investigation locations and methods, including geologic mapping, intended to characterize the rock and rock mass parameters.
	• NRC eRAI 9859 Question 02.05.04-03 revised the following sections:
	  o Section 3.4 to indicate that the specific locations of sensor may be inside and outside of the RB shaft.

Revision Number	Description of Change
	- Section 4.2 to include a description of the calibration process using parameters for the selected soil and rock constitutive models. - Section 5.1 to clarify that the one-step approach is implemented using a linear elastic SASSI analysis approach. - NRC eRAI 9859 Question 02.05.04-04 revised Assumption (1) in Section 5.1.2 and also revised Section 5.1.2 to describe the approach used to account for the effects of anisotropic or heterogenous rock response including potential pressures from unstable blocks of rock mass. - NRC eRAI 9859 Question 02.05.04-05 revised the following sections: - Section 5.1.2 to remove the text stating that isolated unstable blocks do not produce significant loads. - Section 5.1.3 to include other mitigation methods like overexcavation and backfilling when the potential load from a block or wedge is large. - Section 8.0 to include Goodman and Shi (1985) as a reference. - NRC eRAI 9859 Question 02.05.04-06 revised Assumption (4) in Section 5.1.2. - NRC eRAI 9859 Question 02.05.04-07 revised Section 3.1.3 to discuss characterization of the groundwater conditions. - NRC eRAI 9859 Question 02.05.04-08 revised the following sections: - Section 3.1.1 to define d_{max}. - Table 3-1 to correct a typographical error. - Section 3.1.3 to include a discussion on reviewing the current knowledge of the state of stress in the bedrock. - Section 4.3.4.1 to include the influence of groundwater and measured horizontal stresses in the initial stress conditions for the nonlinear FIA. - NRC eRAI 9859 Question 02.05.04-09 revised the following sections: - Section 5.1.4 to explain that the probabilistic earth pressure evaluations also consider uncertainties in the methods or empirical relationships used for development of site related parameters. - Table 5-1 to indicate that the models for probabilistic analyses consider the natural and measurement uncertainties in rock mass properties.

Revision Number	Description of Change
	Added reference citations to the following sections for RG 1.132 and 1.138: 1.1, 2.1, 2.5, 3.0, 3.1, 3.1.1, 3.1.2, 3.1.3, and 3.5. Corrected typographical errors in Sections 1.3, 3.0, 5.1.2, and 5.1.3. Corrected lettered bullets so they start with A in the following sections: 3.2.1, 3.4, 4.1, 4.3, and 7.3.
2	Created “-A” version by adding GEH’s responses to the NRC's Requests for Additional Information (RAIs) (References 8.70 through 8.72) and the NRC’s Final Safety Evaluation (Reference 8.73). Added References 8.70 through 8.73. Made the following editorial corrections: • Section 5.1: 5th paragraph: Corrected “life load” to “live load”. • Section 5.2.1.2: Sentence above Equation 5-22: Corrected “<” to “>”.

Acronyms and Abbreviations

Term	Definition
ABWR	Advanced Boiling Water Reactor
ASME	American Society of Mechanical Engineers
B&PV	Boiler & Pressure Vessel
BTP	Branch Technical Position
BWR	Boiling Water Reactor
COL	Combined Operating License
CP	Construction Permit
DCA	Design Certification Application
DCD	Design Control Document
ESBWR	Economically Simplified Boiling Water Reactor
ESI	Equipment Structure Interaction
ESP	Early Site Permit
FIA	Foundation Interface Analysis
GDC	General Design Criteria
GEH	GE Hitachi Nuclear Energy
HGNE	Hitachi-GE Nuclear Energy Ltd.
ISI	In-service Inspection
LTR	Licensing Topical Report
LWR	Light-Water-Reactor
NPP	Nuclear Power Plant
NRC	Nuclear Regulatory Commission
OGS	Off-Gas System
OL	Operating License
PBSRS	Performance Based Surface Response Spectra
PCV	Primary Containment Vessel
RB	Reactor Building
RG	Regulatory Guide
RPS	Reactor Protection System
RPV	Reactor Pressure Vessel

Term	Definition
RwB	Radwaste Building
SC-I	Seismic Category I
SMAMP	Structures Monitoring and Aging Management Program
SMR	Small Modular Reactor
SRA	Site Response Analysis
SRP	Standard Review Plan
SSCs	Structures, Systems, and Components
SSE	Safe-Shutdown Earthquake
SSI	Soil-Structure Interaction
TB	Turbine Building
TMI	Three Mile Island

1.0 INTRODUCTION

1.1 Purpose

The purpose of this report is to present design, analysis, and monitoring guidelines and requirements for construction of a BWRX-300 Small Modular Reactor (SMR) using innovative and comprehensive approaches that ensures safe operation throughout the life of the plant. The BWRX-300 innovative methodologies and approaches meet 10 CFR 50, Appendix A, General Design Criteria (GDC). Further, the innovative approaches presented herein meet the intent of NUREG-0800 "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition" for large light water reactors and other NRC guidance while addressing specific areas of concern identified in NUREG/CR-7193, "Evaluations of NRC Seismic-Structural Regulations and Regulatory Guidance, and Simulation-Evaluation Tools for Applicability to Small Modular Reactors (SMRs)," (Reference 8.1) for the design of deeply embedded SMRs.

As described in Section 1.3, a cost-effective design concept is implemented for the BWRX-300 where the majority of important safety- related systems and components are located in the below grade Reactor Building (RB) vertical right cylinder shaft. The cost for construction of the BWRX-300 RB is optimized by minimizing the amount of excavation and reducing the amount of backfill, as discussed in Section 1.4. The construction method is provided as relevant background information to identify the effects of deep excavation and construction sequences on site characterization, soil properties, and methodology used to analyze and design the BWRX-300 RB structure, including construction monitoring and inspections.

The following criteria, methodologies, recommendations, and approaches specific to the innovative BWRX-300 design are addressed in the report and may be referenced during future licensing activities either by GEH in support of a 10 CFR 52 Design Certification Application (DCA) or by a license applicant requesting a Construction Permit (CP) and Operating License (OL) under 10 CFR 50 or a Combined Operating License (COL) under 10 CFR 52:

A. Requirements and recommendations are provided in Section 3.1 for site investigation and subsurface materials laboratory testing programs that address the specific BWRX-300 configuration with the RB vertical shaft deeply embedded in in-situ soil and/or rock materials. The provided recommendations are beyond current regulatory guidance for large light water reactors and define additional requirements for characterizing in-situ materials surrounding the deeply embedded SMRs. These additional requirements address current limitations in Regulatory Guide (RG) 1.132 "Site Investigations for Foundations of Nuclear Power Plants," Revision 2 (Reference 8.64), and RG 1.138 "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3 (Reference 8.65), that could adversely affect the results when applied to deeply embedded SMRs, as identified in NUREG/CR-7193, Sections 1.5.3 and 1.5.5.

B. Methodologies and approaches for non-linear Foundation Interface Analyses (FIA) are recommended in Section 4.0 that are supported by the results from the data collected from the inspection and monitoring programs described in Sections 3.2 and 3.3. This innovative approach ensures, with a high level of confidence, that the stability of the deeply embedded BWRX-300 RB structure will be maintained throughout the life of the plant and addresses

specifics related to the design and construction of deeply embedded SMRs. This proposed approach addresses the current limitations of NUREG-0800 Standard Review Plan (SRP) 2.5.4, "Stability of Subsurface Materials and Foundations," Revision 5, and SRP 3.7.1, "Seismic Design Parameters," Revision 4, when applied to deeply embedded SMRs, as identified in NUREG/CR-7193, Sections 1.5.10 and 1.5.11, and is beyond the stability requirements identified in SRP 3.8.5, "Foundations," Revision 4.

C. An innovative field monitoring program approach, supported by the non-linear FIA results, is described in Section 3.4 that serves to: (1) detect possible changes in the properties of in-situ subgrade materials below and around the RB during excavation, construction and operation; and (2) ensures they are enveloped by the BWRX-300 site-specific design. This innovative approach is beyond the current regulatory guidance of SRP 2.5.4 related to the effects on the surrounding soil properties when applied to deeply embedded SMRs, as identified in NUREG/CR-7193, Section 1.5.10.

D. A compressive strength testing program for safety-related concrete is described in Section 3.2.2.1 that ensures the concrete placed during construction of BWRX-300 meets the design specifications. This in-service concrete testing program includes an additional sampling frequency requirement that is beyond the current regulatory guidance related to the volume of safety-related concrete defined in RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," Revision 3, when applied to SMRs, as identified in NUREG/CR-7193, Section 1.5.6.

E. Requirements and recommendations for implementing a one-step approach for static and seismic Soil-Structure Interaction (SSI) analyses are provided in Section 5.1. This one-step approach captures the interaction of the deeply embedded RB structure with the surrounding subgrade and its effects on the RB structural member's design. This section addresses the current limitations in SRP 3.7.1 and SRP 3.7.2, "Seismic System Analysis," Revision 4, related to the interaction of the soil column with the deeply embedded RB structure, as identified in NUREG/CR-7193, Section 1.5.11, and is beyond the current regulatory guidance of SRP 3.8.4, "Other Seismic Category I Structures," Revision 4, and SRP 3.8.5 related to boundary conditions and earth pressure loads.

F. Deterministic and probabilistic evaluation approaches are provided in Sections 5.1.3 and 5.1.4, respectively, which can be used to ensure the one-step approach provides conservative earth pressure design demands on the deeply embedded RB structure by addressing variations in subgrade properties and uncertainties in earth pressure load calculations. These innovative approaches are beyond the current regulatory guidance in: (1) SRPs 3.7.1 and 3.7.2 related to the development and application of inputs used in the analysis and design of deeply embedded SMRs, as identified in NUREG/CR-7193, Section 1.5.11; and (2) SRPs 2.5.4, 3.8.4 and 3.8.5 related to consideration of earth pressure loads.

G. Approaches are recommended in Sections 5.2.1 and 5.2.4, respectively, for developing equivalent linear static and dynamic subgrade properties that are used as inputs to the one-step design analysis model. These approaches ensure that the design of the BWRX-300 RB deeply embedded structure envelopes the uncertainties related to the properties of in-situ soil and rock subgrade materials. These sections address the current limitations in SRP 3.7.1 related to the inputs used for analysis and design of deeply embedded SMRs, as identified in

NUREG/CR-7193, Section 1.5.11, and are beyond the regulatory guidance of SRPs 2.5.4 and 3.8.5.

H. Requirements and methodologies for developing Safe-Shutdown Earthquake (SSE) design ground spectra are provided in Section 5.2.2 that define the design ground motion along the depth of the BWRX-300 RB embedment. These requirements are beyond the current regulatory guidance in DC/COL-ISG-017 "Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses" when applied to deeply embedded SMRs, as identified in NUREG/CR-7193, Section 1.5.8.

I. Two additional requirements are introduced in Section 5.2.3 for generating acceleration time histories for use as input to the seismic SSI analyses that are beyond the current regulatory guidance: (1) the requirement for generating five sets of acceleration time histories that ensures mitigation of uncertainty in the computed responses due to the phasing of the time history frequency components; and (2) the requirement for refining the time step of acceleration time histories to ensure the accuracy of the calculated high-frequency in-structural responses per the guidance of DC/COL-ISG-01 "Final Interim Staff Guidance on Seismic Issues Associated with High Frequency Ground Motion," Section 3.1.1. These additional requirements address the current limitations in SRP 3.7.1 for development of design time histories.

J. Section 5.3 presents a one-step seismic SSI analysis approach that provides demands for seismic design and qualification of structures, systems and components (SSCs) for all frequencies of interest, as described in Section 5.3.2, and adequately captures the effects of structure-soil-structure interaction (SSSI) for the deeply embedded RB with adjacent structures and foundations, as described in Section 5.3.7. This seismic analysis approach addresses current limitations in SRP 3.7.2 when capturing the effects of seismic interaction of the deeply embedded RB structure with adjacent structures through the subgrade, as identified in NUREG/CR-7193, Section 1.5.11.

K. Different approaches are recommended in Section 5.3.4 for demonstrating consistency between the results from the deterministic SSI analyses of the RB structure with the results from the probabilistic site response analyses (SRA) that ensure the motion input used in the seismic SSI analyses is adequate throughout the depth of the RB embedment. These approaches meet the intent of the current regulatory guidance and address current limitations in DC/COL-ISG-017 related to the seismic analysis of deeply embedded structures, as identified in NUREG/CR-7193, Section 1.5.8.

L. Approaches are recommended in Sections 5.3.5, 5.3.8, 5.3.9 and 5.3.10 for sensitivity evaluations from the effects of concrete cracking, soil-structure interface conditions, soil separation and groundwater variations on the seismic response and design of the deeply embedded RB structure. These sensitivity evaluations ensure the seismic design envelopes the variations of these SSI parameters, and address the current limitations in SRP 3.7.2 related to the SSI analysis of deeply embedded SMRs, as identified in NUREG/CR-7193, Section 1.5.11. The sensitivity evaluations are based on comparing the results of the linear SSI analyses for key in-structure responses and structural stress demands. Section 5.3.1 provides guidelines for selection of the key responses used for sensitivity evaluations.

M. A comprehensive approach is recommended in Section 5.3.3 for evaluating the effects of non-vertically propagating seismic waves on the design ground motion and seismic response of the deeply embedded RB structure. This approach is beyond the current regulatory guidance in SRP 2.5.2, "Vibratory Ground Motion," Revision 5, related to the effect of inclined shear-wave propagation on the design of deeply embedded SMRs, as identified in NUREG/CR-7193, Section 1.5.9.

N. Different approaches are provided in Section 5.3.6 for considering Equipment Structure Interaction (ESI) for developing in-structure seismic response demands for equipment design and qualification. These approaches are beyond the current regulatory guidelines in RG 1.122, "Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components," Revision 1, related to ESI effects on design of equipment with more complex dynamic behavior, as identified in NUREG/CR-7193, Section 1.5.2.

O. Recommendations for performing non-linear seismic SSI analyses are presented in Section 5.3.11 for sensitivity evaluations to determine the effects of soil separation and soil secondary non-linearity on the seismic response and design of the deeply embedded RB structures constructed at sites characterized by high seismicity and highly non-linear subgrade materials. These sensitivity evaluations address the current limitations of SRP 3.7.2 related to the SSI analysis of deeply embedded SMRs, as identified in NUREG/CR-7193, Section 1.5.11.

P. Section 6.0 provides a graded approach for the design of structures adjacent to the deeply embedded RB that includes II/I interactions. The approach captures the II/I interaction evaluations under Seismic Category I (SC-I) seismic and extreme wind load conditions and proposes an acceptance criterion of limited inelastic deformations. This section proposes a graded approach for categorization of structures as defined in Regulatory Guide 1.29 "Seismic Design Classification" using a Risk-Informed Performance-Based approach to meet the BWRX-300 design-to-cost goals.

Q. A methodology for developing generic seismic and geotechnical design parameters is presented in Section 7.0 that are used as input for evaluating the applicability of the BWRX-300 generic deeply embedded RB design for a range of conditions present at most North American candidate sites. The methodology presented in this section addresses the regulatory guidance of SRP 2.0, "Site Characteristics and Site Parameters," Revision 1, for selection of site-related characteristics for the design of the deeply embedded BWRX-300 that are representative of a reasonable number of sites.

In summary, the BWRX-300 innovative methodologies and approaches presented herein meet regulation and the intent of the current regulatory guidance for large light water reactors while addressing specific areas of concern related to seismic and structural design of deeply embedded SMRs identified in NUREG/CR-7193 (Reference 8.1). The innovative inspection and monitoring approaches for inspection and monitoring that are supported by the results from non-linear FIA models ensure a greater a level of confidence that the BWRX-300 meets the regulatory guidelines and requirements for maintaining the stability of the RB structure through construction and throughout the life of the plant. The graded approach applied to seismic classification of structures, while fully addressing the II/I interactions of structures adjacent to the RB is in accordance with the regulatory guidance and meets the BWRX-300 design-to-By the cost goals. The seismic and geotechnical design parameters used for the generic design of the BWRX-300 provide a realistic

representation of a variety of different types of conditions at the majority of candidate sites, and are developed following a methodology that is in accordance with the regulatory guidance for developing site-specific design parameters.

The innovative methodologies and approaches presented in Sections 3.0, 4.0 and 5.0 of this report delineate a comprehensive BWRX-300 design process that is illustrated by the flow chart on Figure 1-1. The site investigation and subsurface materials laboratory testing programs are performed following the requirements and recommendations in Section 3.1 to provide the data required for developing:

- Base-case subgrade models for use as input to the probabilistic SRA described in Section 5.2.2
- Equivalent linear static properties for use as input to the static SSI analyses described in Section 5.1
- Non-linear soil and rock constitutive model parameters for use as input to the non-linear FIA and seismic SSI analyses described in Sections 4.0 and 5.3.11, respectively

Linear elastic 1-g static and seismic SSI analyses provide dead, live, seismic inertia and static and seismic earth pressure load demands for the design of the BWRX-300 RB structure. As described in Sections 5.1.3 and 5.1.4, the results of the non-linear FIA are used to ensure the static earth pressure demands used for the RB design envelope uncertainties related to:

- measurements and variations of subgrade material properties; and
- simplifying SSI analysis modeling assumptions described in Section 5.1.2.

The information gathered from the excavation, construction and in-service monitoring programs, described in Sections 3.2, 3.3 and 3.4, is used to calibrate the non-linear FIA model and evaluate any possible changes in the subgrade conditions that can compromise the stability of the BWRX-300 during construction and operation.

As described in Sections 5.2.2, 5.2.3 and 5.2.4, the results of the probabilistic SRA are used for developing design ground motion and strain-compatible subgrade material profiles for use as input to the deterministic seismic SSI analysis. The results of these SSI analyses are used to develop demands for seismic design and evaluation of the BWRX-300 RB SSCs, and as an envelope of results from design basis SSI analyses to address the effects of variation and uncertainties of subgrade properties. An additional set of sensitivity analyses are performed, as described in Section 5.3, to ensure the design envelopes the effects of different SSI parameters. These sensitivity SSI analyses are performed on linear-elastic SSI models representing conditions that bound the variation of the SSI parameters. If the site is characterized by a high seismicity and the results of non-linear static FIA indicate that the non-linear response of subgrade materials is significant, sensitivity SSI analyses may also be performed on non-linear models, as described in Section 6.3.11.

1.2 Scope

The scope of this report includes:

- Regulatory basis specific for the innovative approaches implemented for the analysis, design and construction of the BWRX-300 are described in Section 2.0.
- Guidelines and requirements for characterizing subsurface conditions, including geotechnical site investigations and laboratory testing programs, as well as the inspection and monitoring programs performed during the excavation, construction, and operation of the BWRX-300 are described in Section 3.0.
- Requirements and guidelines for performing FIA to ensure the stability of both structure and the in-situ soil and/or rock during and after construction are described in Section 4.0.
- Design requirements, acceptance criteria and guidelines for the analysis and design of the deeply embedded RB are described in Section 5.0, including the development of site-specific geotechnical and seismic design parameters.
- The BWRX-300 approach for addressing II/I interaction between the SC-I RB and surrounding structures and foundations is presented in Section 6.0.
- Generic seismic and geotechnical design parameters are described in Section 7.0 that ensure the applicability of the BWRX-300 generic design for a range of conditions present at the majority of North American candidate sites.

1.3 Description of the BWRX-300

The BWRX-300 is an approximately 300 MWe, water-cooled, natural circulation SMR utilizing simple safety systems driven by natural phenomena. It is being developed by GE Hitachi Nuclear Energy (GEH) in the USA and Hitachi-GE Nuclear Energy Ltd. (HGNE) in Japan. It is the tenth generation of the BWR. The BWRX-300 is an evolution of the U.S. NRC-licensed, 1,520 MWe Economic Simplified Boiling Water Reactor (ESBWR). Target applications include base-load electricity generation and load-following electrical generation.

An innovative design-to-cost solution has been developed for the BWRX-300 to optimize the construction cost and schedule and maximize its safety performance during the operational and decommissioning life of the plant. The BWRX-300 Reactor Pressure Vessel (RPV), Pressure Containment Vessel (PCV) and other important safety-related systems and components are located in the below-grade RB vertical right-cylinder shaft to mitigate effects of possible external events, including aircraft impact, adverse weather, flooding, fires, and earthquakes. Fuel handling equipment and pools containing water needed for the BWRX-300 passive safety-related cooling systems are in the above-grade portion of the RB directly supported by the below-grade vertical shaft. Advanced construction methods are used for the RB below-grade vertical shaft to reduce the construction cost and schedule by minimizing the amount of excavation, concrete, and the use of engineered backfill materials.

Figure 1-2 shows a conceptual site plot plan for the BWRX-300 single unit plant. This drawing is provided for information only and may not reflect the final BWRX-300 site-specific design. As shown on Figure 1-2, Control Building (CB), Turbine Building (TB) and Radwaste Building (RwB) structures that are supported by near-surface basemat foundation are located adjacent to the

deeply embedded SC-I RB structure. CB, TB and RwB are separated from the RB by seismic gaps. The CB houses the control room, electrical, control and instrumentation equipment. RwB houses rooms and equipment for handling, processing, and packaging liquid and solid radioactive wastes. TB encloses the turbine generator, main condenser, condensate and feedwater systems, condensate purification system, off-gas system (OGS) cooler and refrigerant dryer, turbine-generator support systems and bridge crane.

The RwB, which houses the systems for management of radioactive gas, liquid and solid radiological waste is categorized as RW-IIa in accordance with Regulatory Guide (RG) 1.143 “Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants,” Revision 2.

The CB, TB and RwB structures are evaluated as described in Section 7.0 to prevent structural failure or interaction that could:

- degrade the functioning of the RB SC-I SSCs to an unacceptable level of safety;
- result in incapacitating injury to occupants of the CB control room; and
- compromise the safety functions of those SSCs that are required to remain functional following a seismic event.

Figure 1-3 shows a conceptual RB section view for the BWRX-300. This drawing is provided for information only and does not reflect the final BWRX-300 design.

1.4 Reactor Building Below-Grade Shaft Construction

The construction process determines the final configuration of the interface between the RB below-grade shaft and surrounding soil and/or rock and defines the scope of the analysis and design of the RB. The construction method also influences the scope and extent of subsurface site investigations in addition to the inspections and monitoring requirements during construction and throughout the life of the plant through decommissioning.

For most subsurface conditions, traditional methods for excavation and construction of the BWRX-300 RB translate into prolonged schedules and high costs. To optimize the cost of the BWRX-300, innovative approaches are employed for the construction of the RB below grade shaft that are aimed to:

- minimize the amount of excavation;
- reduce the amount of engineered backfill; and
- reduce construction schedule.

The construction approach for the BWRX-300 will require adaptation to the site-specific conditions. Most physical subsurface settings will consist of configurations that include soil overburden and rock beneath. For purposes of illustrating the construction approaches, a generic transition profile is considered having the upper two thirds with soft soil strata followed by a lower third with hard rock strata. This transition profile is selected to include the required transition in construction techniques from soil to rock subgrade conditions.

The Open Caisson construction method, shown on Figure 1-4, is applied as the preferred approach for the construction of the BWRX-300 RB shaft. This method is used to leverage the excavated shaft wall as formwork for the outer RB wall face and construct the RB from the shaft bottom upwards. A circular slurry shoring wall is installed in the softer upper soil strata and socketed into bedrock to stabilize the excavation. The shaft would continue to be excavated through rock down to the bottom of basemat, exposing the surface of the rock face for inspections. As shown on Figure 1-4, waterproofing would be applied to the surface of the slurry wall and the rock face, which in turn be used as formwork for the outer surface of the below-grade RB shaft.

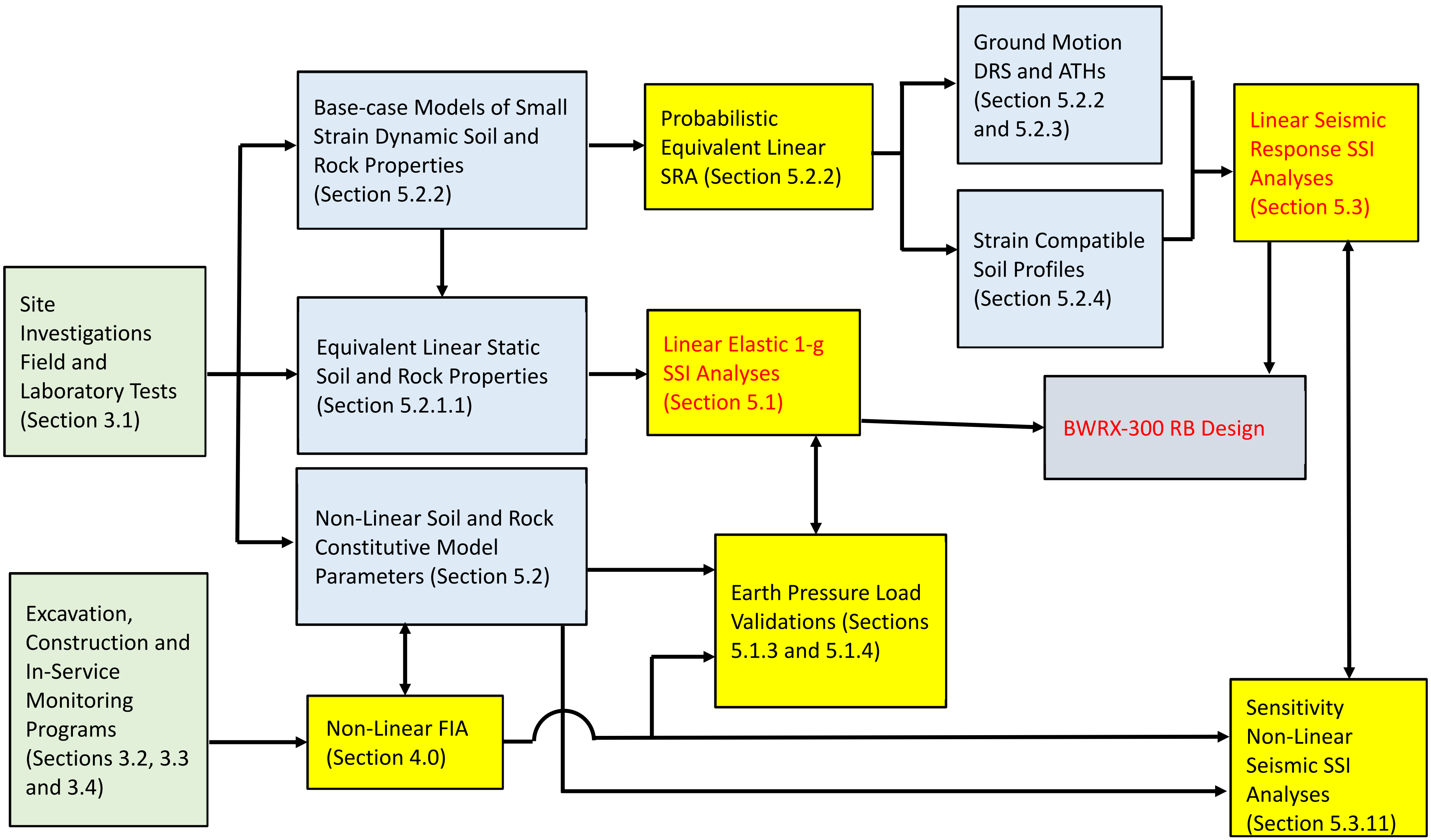


Figure 1-1: BWRX-300 Investigation, Monitoring, Analysis and Design Process

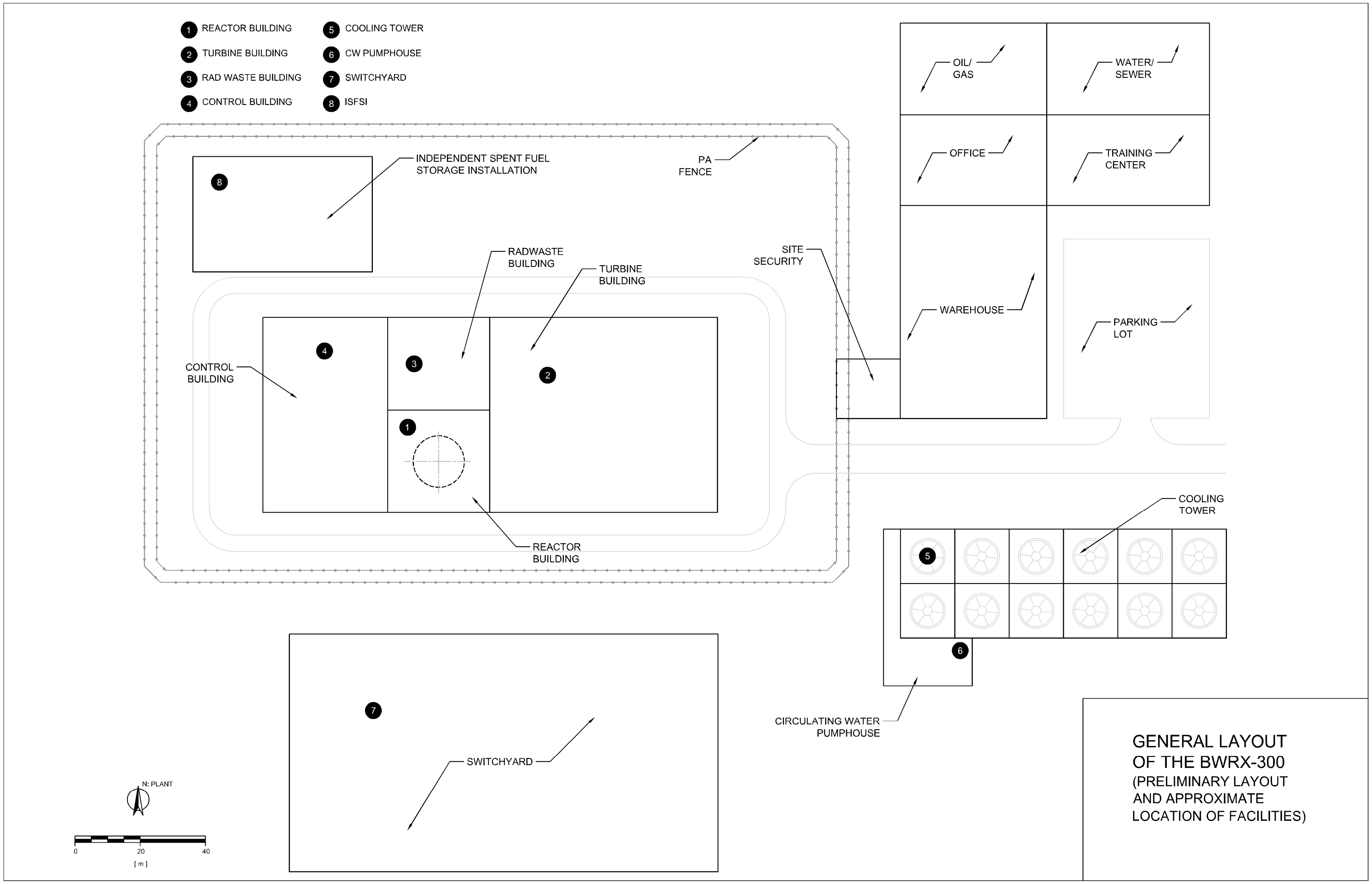


Figure 1-2: BWRX-300 Conceptual Site Plan

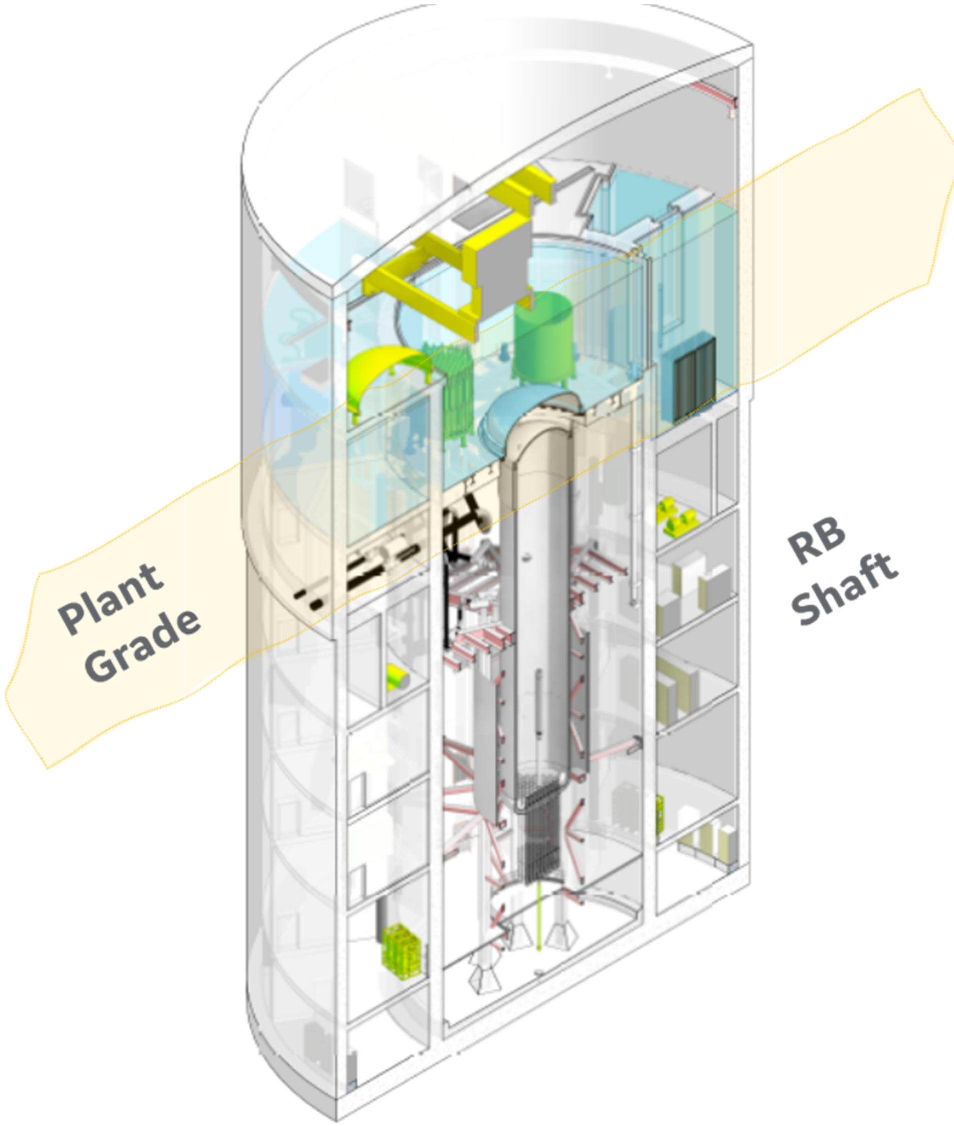


Figure 1-3: BWRX-300 Conceptual RB Three-Dimensional Section View

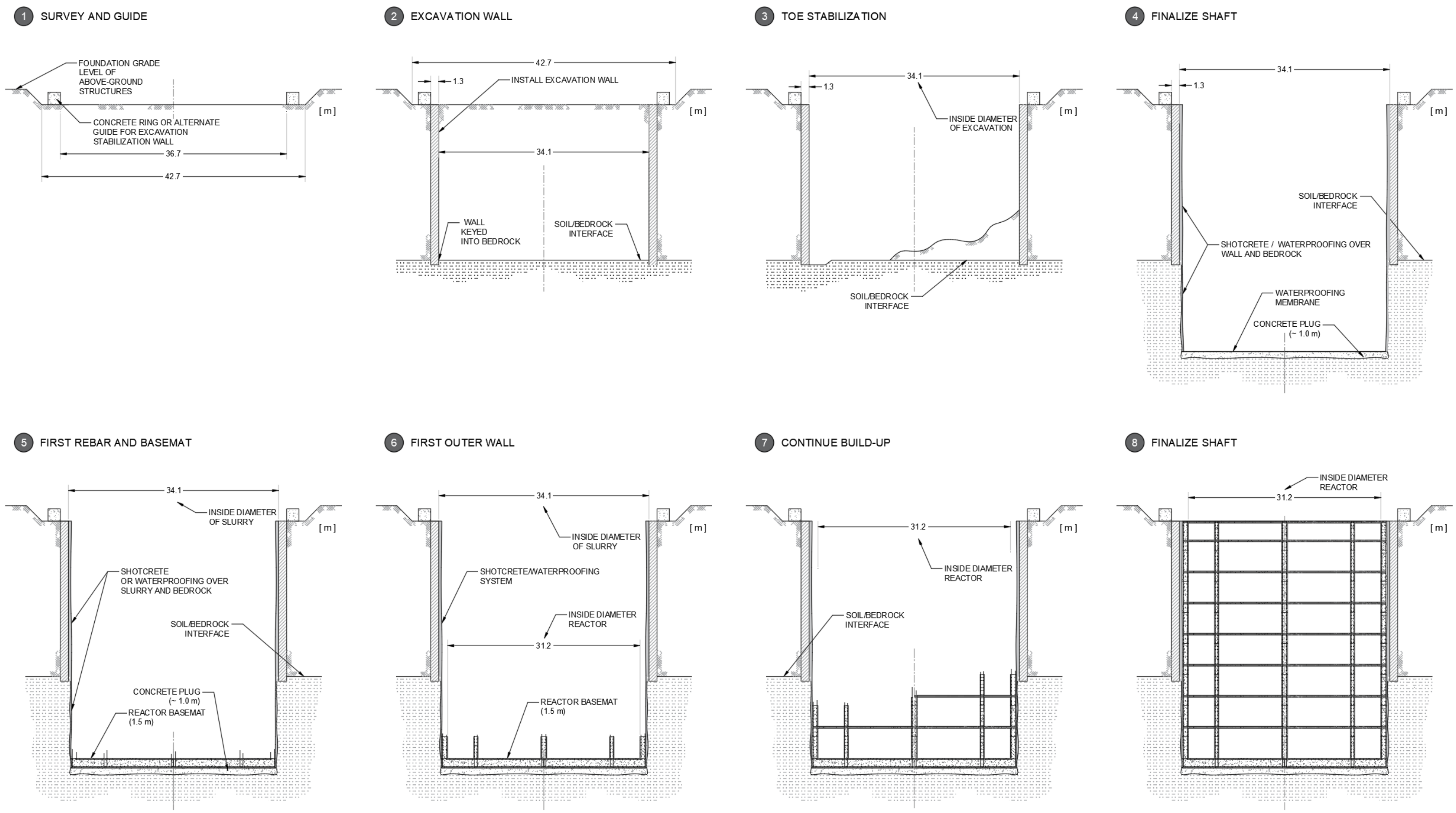


Figure 1-4: BWRX-300 Reactor Building Shaft Construction

2.0 REGULATORY BASIS

This section describes compliance with regulatory requirements and the bases for any exemptions to regulatory requirements or approaches to regulatory guidance that may be referenced during future licensing activities either by GEH in support of a 10 CFR 52 Design Certification Application (DCA) or by a license applicant requesting a Construction Permit (CP) and Operating License (OL) under 10 CFR 50 or a Combined Operating License (COL) under 10 CFR 52.

10 CFR 50, Appendix B establishes quality assurance requirements for the design, manufacture, construction, and operation of nuclear power plant SSCs that prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public. The pertinent requirements of 10 CFR 50 Appendix B apply to all activities affecting the safety-related functions of those SSCs.

10 CFR 50, Appendix B establishes in Clauses X and XI quality assurance requirements for the design, construction, and operation of nuclear power plant SSCs that prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public.

The following is the regulatory basis specific to the innovative approaches implemented for the analysis, design, construction, and maintenance of the BWRX-300 important to safety structures. The implemented innovative approaches meet the intent of the current regulatory guidance for large light water reactors and address the specifics related to the seismic and structural design of deeply embedded SMRs identified in NUREG/CR-7193 (Reference 8.1).

2.1 Regulatory Basis for Defining Site Subsurface Conditions

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

IAEA Safety Guide NS-G-6 provides guidance on the methods and procedures for analyses to support the assessment of the geotechnical aspects for the design of nuclear power plants.

SRP 2.5.4 provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Revision 2 (Reference 8.64), describes methods acceptable to the NRC staff for conducting field investigations to acquire the geological and engineering characteristics of the site and provides recommendations for developing site-specific guidance for conducting subsurface investigations. Details regarding the technical bases described in RG 1.132 are provided in NUREG/CR-5738 "Field Investigations for Foundations of Nuclear Power Facilities" (Reference 8.2).

RG 1.138 (Reference 8.65) describes laboratory investigations and testing practices for determining soil and rock properties and characteristics needed for engineering analysis and design of foundations and earthworks for nuclear power plants. NUREG/CR-5739 "Laboratory Investigations of Soils and Rocks For Engineering Analysis and Design of Nuclear Power Facilities" (Reference 8.3), provides the technical basis for the techniques for laboratory testing of

soils and rock described in RG 1.138, including summaries of the processes for a laboratory testing program, ranging from storage, selection, handling of test specimens to static and dynamic testing methods and equipment.

Section 3.1 provides a general description of the site investigation and laboratory testing programs implemented for the BWRX-300 that are compliant with the regulatory guidance of SRP 2.5.4, RG 1.132, and RG 1.138. Besides the techniques listed in RG 1.132, Appendix C, the BWRX-300 site investigation program may use other advanced technologies described in Section 3.1 for detection and mapping of joints, seams, cavities, and other features in the excavated rock face.

Guidelines for the spacing and depth of the BWRX-300 specific subsurface investigations are provided in Section 3.1.1. These guidelines are beyond those specified in Appendix D of RG 1.132 to ensure the site investigation results provide adequate information to define the static and dynamic engineering properties and the spatial distribution of soil and rock materials surrounding the BWRX-300 RB.

Section 3.1.3 provides recommendations for the site investigation to effectively characterize rock discontinuities such as cavities, fracture zones, joints of the weathered rock layers and inclined bedding planes between different rock formations that may affect the structural integrity of the RB below-grade shaft.

Section 3.2, 3.3, and 3.4, in combination with Section 4.0, present an innovative approach for monitoring the effects of excavation and construction on the properties of subsurface materials per regulatory guidance of SRP 2.5.4, with emphasis in Subsections 2.5.4.3, 2.5.4.5, 2.5.4.6, and 2.5.4.10.

2.2 Regulatory Basis for Development of Site Design Parameters

10 CFR 50 Appendix A, General Design Criteria (GDC) 2, Design bases for protection against natural phenomena, requires that nuclear power plant SSCs important to safety be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The bases for design of important to safety SSCs is determined by the SSCs safety functional requirements.

10 CFR 100.23(d)(1) specifies the requirements for defining the safe shutdown earthquake (SSE) ground motion for the site and the need for addressing result uncertainties in the site investigation performed as noted in Section 2.1.

SRP 3.7.1 provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

RG 1.208, “A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion,” Revision 0, specifies the performance-based approach Chapters 1 and 2 of ASCE/SEI 43-05 (Reference 8.4) standard as an acceptable approach for defining the SSE Ground Motion Response Spectra (GMRS) that satisfies the requirements of 10 CFR 100.23. The performance-based site-specific SSE spectra is defined based on the results of a site Probabilistic Seismic Hazard Analysis (PSHA) following the provisions of Chapter 2 of ASCE/SEI 43-05 (Reference 8.4) standard. The Approach 3 methodology, described in NUREG/CR-6372 “Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use

of Experts" (Reference 8.5), is used to account for the wave propagation characteristics of the site and to address uncertainties related to the site sub conditions and development of:

- Foundation Input Response Spectra (FIRS) defining the amplitude and frequency content of the design ground motion at the foundation bottom; and
- Performance Based Surface Response Spectra (PBSRS) defining the amplitude and frequency content of the design ground motion at the ground surface.

Interim Staff Guidance (ISG) DC/COL-ISG-017 "Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses" (Reference 8.6), specifies the requirements for ensuring the inputs used for the deterministic SSI analysis of embedded structures are consistent with the results of probabilistic SRA used to develop FIRS and PBSRS using NUREG/CR-6372, Approach 3 (Reference 8.5).

Section 5.2.3 describes an approach for development of ground motion acceleration time histories (ATHs) that is in accordance with the regulatory guidance of SRP 3.7.1 and meets the additional requirements of Section 2.6 and 4.2.1(b) of ASCE/SEI 4-16 "Seismic Analysis of Safety-Related Nuclear Structures" (Reference 8.7).

Section 5.2.4 describes the methodology for development of subgrade profiles of hazard constituent dynamic properties compatible to the strains generated by an SSE level seismic event from the results of Approach 3 probabilistic site response analyses that is in accordance with Section 2.4 and 5.1.7 of ASCE/SEI 4-16 (Reference 8.7). Section 5.2.1 describes the approach used for development of equivalent linear static properties of in-situ soil and rock material for use as input for the design including a description of the approaches for addressing non-linear aspects of subgrade material behavior in the design that is based on results of linear elastic analyses.

Besides the approaches recommended in Section 5.2 of DC/COL ISG-017 (Reference 8.6), alternative approaches are described in Section 5.3.4 for ensuring the BWRX-300 seismic design meets the DC/COL ISG-017 guideline for consistency between the results of deterministic SSI analyses with results from the probabilistic site response analyses. The consistency between free field motion for the deterministic SSI analysis of the deeply embedded BWRX-300 RB structure and probabilistic SRA is checked not only at the ground surface using the PBSRS, but also at intermediate elevations along the embedment depth using Performance Based Intermediate Response Spectra (PBIRS). These PBIRS that define the outcrop free field motion at selected elevations corresponding to significant shear-wave velocity (V_S) contrasts in the SSI analysis subgrade profiles, are developed using the Approach 3 methodology consistent with the methodology used for development of FIRS at the foundation bottom elevation.

2.3 Seismic Analysis Regulations

10 CFR 50 Appendix S, Earthquake Engineering Criteria for Nuclear Power Plants, requires that SSCs that shall be designed to withstand the effects of the SSE ground motion or surface deformation are those necessary to assure: (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shut down the reactor and maintain it in a safe-shutdown condition, and (3) the capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 50.34(a)(1) or 10 CFR 100.11.

The basis for the seismic design of BWRX-300 RB SSCs is developed based on the results of SSI analyses performed following the regulatory guidance of SRP 3.7.2 and DC/COL ISG-01 (Reference 8.8), and in accordance with the ASCE/SEI 4-16, Section 5 provisions (Reference 8.7).

The SSI analyses are performed on finite element (FE) models of RB that are developed in accordance with the regulatory guidance of SRPs 3.7.1 and 3.7.2, and RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," Revision 1, and the provisions of Section 3 of ASCE/SEI 4-16 (Reference 8.7). Section 5.1 describes the implementation of a one-step approach that addresses Section 3.1.2 of ASCE/SEI 4-16 for the BWRX-300 design that directly uses the results of the SSI analysis FE model as input for the design of the RB structural members.

Following the provisions of Section 5.1.5 of ASCE/SEI 4-16, the effects of structure-soil-structure interaction (SSSI) of R/B with surrounding foundations are incorporated in the design of RB SSCs as described in Section 5.3.7. Dynamic properties of subsystems, components, and equipment are included in the SSI analysis model based on the decoupling criteria of SRP 3.7.2 Subsection II.3.B, considering the effects of ESI as described in Section 5.3.6.

Per the requirements of ASCE/SEI 4-16, Section 5.1, the effects of non-vertically propagating seismic waves, soil separation, concrete cracking and soil secondary non-linearity on the seismic response and design of BWRX-300 RB are evaluated based on responses obtained from linear elastic and non-linear sensitivity SSI analyses described in Sections 5.3.3, 5.3.5, 5.3.8, 5.3.9, 5.3.10 and 5.3.11.

2.4 II/I Interaction Regulations

For the structures adjacent to the RB, the regulatory guidance of SRP 3.7.2 Subsection I.8, related to the requirements of interaction between Non-SC-I structures with SC-I SSCs structures that are referred to by the industry term "II/I interactions" is used. SRP 3.7.2 Subsection II.8 provides the following three II/I interaction criteria for which each non-SC-I structure should meet at least one:

A. The collapse of the non-SC-I SSC will not cause the non-SC-I SSC to strike a SC-I SSC.

B. The collapse of the non-SC-I SSC will not impair the integrity of SC-I SSCs, nor result in incapacitating injury to control room occupants.

C. The non-SC-I structure is analyzed and designed to prevent its failure under SSE conditions.

SRP 3.3.2, "Tornado Loadings," Revision 3, Subsection II.4 requires prevention of similar II/I interactions due to tornado loading so that failure of any structure or component not designed for tornado loads will not affect the capability of other SSCs to perform necessary safety functions. Because SC-I structures are designed for extreme wind conditions (tornado and/or hurricane), II/I interaction evaluations are performed for SSE as well as extreme wind loading.

As described in Section 6.0, the structural members of the CB, RwB and TB resisting horizontal loads are checked to ensure they can satisfy Criterion C so their collapse under extreme environmental design conditions, SSE and tornado and extreme wind loads, is prevented. The design also ensures that under these extreme environmental design conditions, the CB structure does not collapse to result in incapacitating injury to the control room occupants.

The II/I seismic interaction checks are performed considering limited inelastic responses in accordance to the provisions of ASCE/SEI 43-05 (Reference 8.4) and the governing design codes described in Sections 6.2 and 6.3, respectively.

2.5 Testing, Inspection and Monitoring Regulations

10 CFR 50 Appendix A, GDC 1, Quality standards and records, requires that important to safety structures be constructed and tested to quality standards commensurate with the importance of the safety functions to be performed. RG 1.142 and RG 1.136, "Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments," Revision 3, provide guidance for testing of safety-related nuclear concrete structures and concrete containments, respectively.

To meet inspection and testing requirements of 10 CFR 50 Appendix A, GDC 1, BWRX-300 construction inspection and testing programs satisfy:

- the geotechnical and foundation requirements of the NRC Inspection Manual 88131;
- the structural concrete activities requirements of the NRC Inspection Manual 88132; and
- the structural welding inspection requirements of NRC Inspection Manual 55100.

10 CFR 50.65, Requirements for monitoring the effectiveness of maintenance at nuclear power plants, specifies the maintenance rules for monitoring the performance or condition of structures against established goals to provide reasonable assurance that these structures can fulfill their intended safety functions. RG 1.160, "Monitoring the Effectiveness of Maintenance at Nuclear Power Plants," Revision 4, and NUMARC 93-01 "Industry Guideline for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants" (Reference 8.9) provide regulatory guidance for demonstrating compliance with the provisions of 10 CFR 50.65.

To meet the requirements of 10 CFR 50.65(a)(1), a monitoring program is established for a periodic assessment of the extent and rate of degradation of basemats and below grade exterior walls in accordance with the regulatory guidance of Section 1.3 of RG 1.160 and Sections 9.4.1.4 and 10.2.3 of NUMARC 93-01. Per Section 9.4.2.4 of NUMARC 93-01, the monitoring program can include non-destructive examinations, visual inspections, vibration, deflection, thickness, corrosion, or other monitoring methods.

Section 3.1 describes techniques for assessment of the site-specific soil and rock investigations following the guidance of RG 1.132 (Reference 8.64) and RG 1.138 (Reference 8.65).

Section 3.1.3 describes techniques to quantitatively characterize the geologic and engineering rock mass properties following the guidance of RG 1.132 and NUREG/CR-5738 (Reference 8.2).

Section 3.2 describes the construction inspection and monitoring program required during construction following the guidance of RG 1.132, including the requirements of NUREG/CR-5738 (Reference 8.2), Appendix A and B. During the BWRX-300 construction, a concrete compressive strength testing program is performed in accordance with RG 1.142 as described Section 3.2.2.

Section 3.3 describes the implementation of monitoring structures in accordance with 10 CFR 50.65, the guidance of RG 1.160 and NUMARC 93-01 for monitoring subsurface conditions to ensure any changes during the operational life of the BWRX-300 are bounded by the design.

3.0 INVESTIGATIONS, TESTING, INSPECTION AND MONITORING PROGRAMS

Due to a significant portion of the RB structure being embedded in in-situ subgrade materials, innovative approaches are required to characterize the in-situ subgrade properties and monitor the subgrade conditions. The interaction of the structure with the surrounding soil and rock is an important factor for the integrity of the RB structure and its response under static and dynamic loads. The changes of stress regimen in the subgrade materials during the excavation, construction, and operation of the BWRX-300 are manifested in the movement of soil and rock below and around the excavation. The dewatering scheme used during the excavation and construction as well as changes in ground water level during the operation of the plant may also affect the subgrade conditions. Furthermore, due to the shaft construction technique implemented, the traditional methods of visual inspections during construction and operation are restricted due to limited accessibility requiring innovative approaches for monitoring the stability of the BWRX-300.

The BWRX-300 suitability for a particular site is verified prior to deployment by an extensive site investigation program as described in Section 3.1. Requirements and recommendations are provided in Section 3.1 for site investigations that are beyond the current regulatory guidelines for large LWRs that address specifics related to the design and construction of the deeply embedded RB identified in NUREG/CR-7193, Sections 1.5.3 and 1.5.5 (Reference 8.1).

A compressive strength testing program of safety-related concrete is described in Section 3.2.2.1 that is beyond the current regulatory guidance of RG 1.142. An additional requirement is adopted for the BWRX-300 testing program that ensures a sufficient number of tests is performed and addresses the specifics of construction of smaller SMRs identified in NUREG/CR-7193, Section 1.5.6 (Reference 8.1).

As noted in NUREG/CR-7193, Section 1.5.10 (Reference 8.1), the effects of the deep excavation, construction and dewatering on subgrade properties may be more significant for the stability of deeply embedded SMRs than for large reactors. Therefore, the monitoring of BWRX-300 site conditions continues throughout the remainder of BWRX-300 excavation, construction, and operation. Construction and in-service monitoring programs, described in Sections 3.2 and 3.3, are implemented to ensure the as built conditions are within the design estimates.

Field instrumentation, described in Section 3.4, is used to monitor subgrade movements and groundwater level changes.

Information gathered from the field instrumentation is used to calibrate the non-linear FIA, described in Section 4.0, and evaluate any possible changes in the subgrade conditions that could potentially compromise the stability of the BWRX-300 during construction and operation. The changes in the subgrade properties are examined through constitutive models, described in Section 4.2, that are used for mathematically representing the behavior of soil and rock materials in the non-linear FIA.

Investigation, testing, inspection, and monitoring programs in conjunction with the results of a set of FIA described in Section 4.3.4, ensure the safe siting of the BWRX-300 plant throughout the following stages:

a) ***Site Characterization***: comprehensive site investigation and laboratory testing programs are employed for the BWRX-300 that are beyond the recommendations in RG 1.132 (Reference 8.64) and RG 1.138 (Reference 8.65) to address specifics related to the design and construction of the deeply embedded RB. Section 3.1 provides requirements and recommendations for the BWRX-300 site investigations and subsurface material laboratory testing programs.

b) ***Excavation***: the removal of in-situ subgrade materials and dewatering of the excavation alters the initial stress and deformation site conditions that may influence the properties of the in-situ subgrade materials. The excavation may cause heave at the shaft bottom and displacements at the shaft sides that may result in changes of the properties and response of the soil and rock materials. The groundwater level and soil/rock movement are closely monitored in order to detect possible instabilities of subgrade materials during the excavation. As discussed in Section 4.3.4, comparisons between the results of FIA and monitoring program may be performed to assess, if the excavation behaves within the realm of expectations. Furthermore, the excavation stage provides the opportunity to inspect exposed surfaces of rock prior to construction. These inspections follow the guidance in RG 1.132 and Appendices A and B of NUREG/CR-5738, as discussed in Section 3.2.1.

c) ***Construction***: a successful construction process must be ensured because it may have a direct implication on the safety of the BWRX-300. The stability of the RB structure, foundation, and surrounding soil and rock are analyzed throughout the various construction stages by comparing data collected from the construction inspections and field observations, described in Section 3.2.2 and 3.4, respectively, with responses obtained from FIA, described in Section 4.0. Any changes in subgrade conditions during the construction process are evaluated and incorporated into the FIA models as discussed in Section 4.3.4.

d) ***Loading***: the behavior of the structure may be critical during loading stages that include the weight of fuel, water in the pools, and other permanent loads that were not previously introduced during construction. The field monitoring program and FIA modeling, described in Sections 3.4 and 4.3.4, continue through this stage and are used to confirm that the response of the subgrade and the RB structure to the additional permanent loads meets the safety requirements.

e) ***Start-up and Operation***: during this stage, two aspects are modeled and monitored by in-service and field monitoring programs described in Sections 3.3 and 3.4. The first is the continued monitoring of settlement and groundwater changes. Depending on soil types, long-term settlement response may be anticipated. Even if long-term settlement is not anticipated, this condition will require confirmation from monitoring. The second aspect relates to external events. Examples are forces from design ground motion, pressures and hazards from design flood, and potential subsurface deformation that originate from instabilities like undetected subsurface conditions or rock cavities, among others. As

described in Section 4.3.4, sensitivity studies may be performed to analyze potential formation of instabilities in the subsurface or to investigate the effects of flooding.

3.1 Site Investigation and Subsurface Material Testing Programs

The soil and rock properties and profiles for static and dynamic SSI analyses are established based on in depth site-specific investigations, field and laboratory testing programs described in Sections 3.1.1 and 3.1.2, following the guidance of RG 1.132 (Reference 8.64) and RG 1.138 (Reference 8.65). The intent of the guidance provided herein is to ensure adequate site investigation and subsurface material testing programs to yield the necessary inputs for the non-linear FIA described in Section 4.0, the probabilistic SRA described in Section 5.2.2, and the development of subgrade properties for the static and seismic design SSI analysis as described in Sections 5.2.1 and 5.2.4, respectively. Section 3.1.3 presents approaches for characterization of the rock mass properties based on the results of site investigation and testing programs.

Site characterization satisfies the guidelines presented in RG 1.132, including the guidance of NUREG/CR-5738, Appendices A and B. Beside the subgrade materials supporting the RB foundation, augmentation of the site investigation guidelines of RG 1.132, and specifically NUREG/CR-5738 Appendix B, are required for deeply embedded structures to provide a full characterization of the extent of in-situ materials surrounding the embedded RB shaft, including establishing an appropriate number, type, and extent of in-situ tests, such as borings, geophysical tests, and groundwater monitoring.

The quality and amount of site-retrieved data dictates the levels of epistemic uncertainty and aleatory variability that needs to be accounted for in the analysis and design of the BWRX-300. The design is therefore tied to a comprehensive site-specific investigation that follows the guidance of RG 1.132 and RG 1.138. The design of laboratory testing investigations depends on findings from the field activities and therefore a more in-depth discussion is omitted in this report.

Site investigation and subsurface material testing programs are recommended herein that are beyond the current regulatory guidance:

- address additional requirements specific to the innovative approaches implemented for the design and construction of the deeply embedded BWRX-300 RB in in-situ subgrade materials;
- ensure the design envelopes possible changes in the subsurface conditions during the excavation, construction, and operation of the BWRX-300 plant; and
- ensure the integrity of BWRX-300 structures are not compromised during the construction and operation.

The recommendations provided herein meet the minimum requirements for a generic greenfield candidate site. The actual number and types of field and laboratory tests are dictated by the site-specific conditions, such as the types of subgrade materials present at the site and their variation. For previously investigated sites, such as sites with issued Early Site Permits (ESP), the number of field and laboratory tests will likely be a subset of the recommendations provided in this report. At such locations, additional investigations and tests will be narrowed to close information gaps between the existing available information and specific needs for the siting of the BWRX-300.

3.1.1 Site Investigation Program

Figure 3-1 represents a preliminary layout of the BWRX-300 footprint and facilities with the deeply embedded RB being the only SC-I structure in the BWRX-300 plant. It is common practice to perform borings and tests below the footprint of the SC-I facilities and to deeper depths than the basemat (RG 1.132, Reference 8.64). The excavation approach minimizes the use of engineered backfill materials as well as the deployment depth of the BWRX-300 RB and requires a subsurface investigation that covers areas beyond its foundation perimeter.

When bedrock units are anticipated to be encountered at depths for engineering purposes, geologic mapping of outcrops should be completed prior to finalizing the number, orientations, and locations of the field investigation borings and tests. This geologic mapping is intended to characterize the anticipated rock mass, discontinuities and to allow for modification of the field investigation to collect appropriate data near the RB shaft.

The diameter of the RB SC-I footprint is relatively small when compared to footprints of typical conventional nuclear plants. The characterization of a small portion of the subsurface environment would be insufficient to adequately characterize the variations and uncertainties in the site subsurface conditions and provide inputs for the Approach 3 probabilistic SRA described in Section 5.2.2. Tests, such as seismic refraction or reflection studies that are useful to map bedrock or detect potential voids become meaningful and possible only when covering greater areas. Measurements of shear-wave velocities (V_S) and compression-wave velocities (V_P) are not sufficient to characterize lateral variability if these are made just a few meters apart.

In order to address the specific requirements of the BWRX-300 RB design, the subsurface site investigations are performed following the guidelines of RG 1.132 for SC-I type site investigations considering the combined footprint areas of the RB SC-I foundation and the adjacent TB, CB and RwB foundations. The extended area considered by the BWRX-300 subsurface site investigation ensures an adequate characterization of the subsurface conditions under the TB, RwB and CB foundations and resulting surcharge loads, which are important for the design of the deeply embedded RB structure and seismic design of RB SC-I SSCs.

Appendix D of RG 1.132, Spacing and Depth of Subsurface Explorations for Safety-Related Foundations, specifies the need for at least one boring underneath each projected safety-related structure or 1 boring for each 900 m^2. The footprint of the main containment shaft and the above ground surrounding structures is about 1 Ha (10,000 m^2). This implies that at least 10 borings would be required for the site investigation. RG 1.132 indicates that the boring depth should go past "the maximum required depth for engineering purposes." If bedrock is encountered, then the boring should penetrate past zones of weakness that could affect foundation performance and extend at least 6 m into sound rock. For the BWRX-300, the maximum required depth for engineering purposes d_{max} is set at approximately 120 m, a depth that is the greater than the following:

a) The depth of the shaft plus twice the diameter of the shaft, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition, and

b) Twice the width of the plant's footprint, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition.

The extent and detail of the site investigation depend on the encountered subsurface conditions. Table 3-1 lists the expected types and amounts of tests that are required to properly characterize site conditions. A boring and geophysical exploration layout is given on Figure 3-1. Table 3-2 lists the recommended borings and their purpose. A minimum of 21 boring locations is anticipated within the BWRX-300 site investigation program which exceeds the minimum of 10 borings based on recommendations in RG 1.132, Appendix D. The increase in the number of borings is to ensure adequate characterization of subsurface properties under and around the deeply embedded RB structure. Previously investigated sites may have information that cover a wide area. In such cases, only limited and targeted additional exploration points may be required.

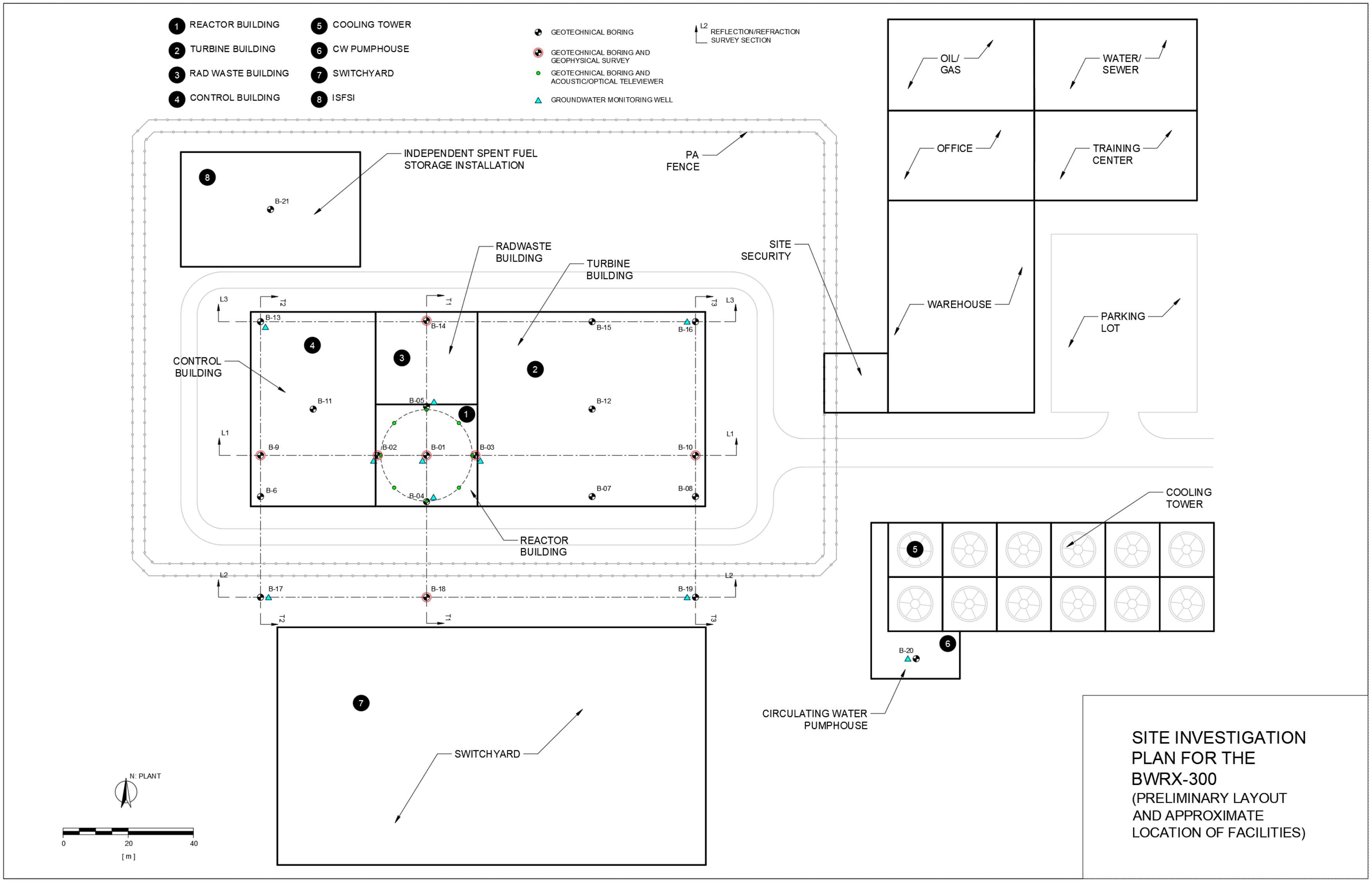


Figure 3-1: Preliminary Layout of the BWRX-300 Borings

Table 3-1: Site Investigation for the BWRX-300

Test Type		Test Purpose	Number of Tests [1]
1	Geotechnical borings	− Measure Standard Penetration (SPT) − Measure Cone Penetration Resistance − Sample soils and rock for visual classification and laboratory testing − Rock Quality Designation (RQD) − Characterize rock mass and discontinuities − Perform pressuremeter tests on weak to moderately soft rock portions to have data parameter for estimation of elastic moduli − Measure in-situ stress (overcoring, hydraulic fracturing)	− 3 borings at perimeter and center of containment down to 120 m − 2 borings at perimeter of containment down to a depth of 60 m − About 18 additional borings designed to cover the footprint of the main facilities, meet the regulatory guidance, and characterize the subsurface as a unit. (see Figure 4-1)
2	Wells	− Groundwater characterization (pump and slug tests, baseline groundwater quality) − Characterize groundwater flow direction and quantify hydraulic gradients	− 9 wells at the center and edge of containment to anticipated depth of 60 m − 4 wells down to a depth of 60 m covering the footprint of the facility
3	Geophysical boring	− Measure V_P and V_S with at least two methods: seismic downhole survey, crosshole, and/or and PS Log suspension survey.	− One boring down to 120 m at center − 4 borings at perimeter of containment down − 4 borings located a distance apart from RB to allow for wider cross sections and correlations to refraction or reflection surveys
4	Refraction Survey	− For sites in which a bedrock horizon is identified by the boring program, perform seismic refraction to obtain a three-dimensional mapping of the bedrock horizon and the thickness of weathered layers	− One grid of surveys covering the footprint extension of the facility
5	Seismic reflection survey	− Identify if voids, sinkholes, karst, or faults are present beneath the footprint of the facilities	− Three longitudinal and two to three transverse reflection sections
6	Borehole Televiewer (Optical/Acoustic)	− Observe rock surface directly, subsurface lithology and structural features such as fractures, fracture infillings, foliation, and bedding planes. − Measure orientation and spacing of rock discontinuities − Packer water-pressure tests in rock − Measure in-situ stress (overcoring, hydraulic fracturing)	− Relevant for rock conditions, over which boring recovery and RQD allow for an open borehole. − The proposed 8 televiewer locations will support a better characterization of the rock mass and as a substitute for potential inspection limitations due to the construction process.
[1] Number may be adjusted depending on encountered site conditions and site available information			

Table 3-2: Anticipated Boring Program

Boring	Depth [(1)] (m)	SPT[(2)]/ CPT[(3)] Coring	VEL DH[(1)]	VEL PS LOG[(2)]	Well
B-01	120	✓	✓	✓	✓
B-02	120	✓	✓	✓	✓
B-03	120	✓	✓	✓	✓
B-04	60	✓			✓
B-05	60	✓			✓
B-06	60	✓			
B-07	30	✓			
B-08	60	✓			
B-09	80	✓	✓	✓	
B-10	80	✓	✓	✓	
B-11	60	✓			
B-12	60	✓			
B-13	80	✓			✓
B-14	80	✓	✓	✓	
B-15	60	✓			
B-16	80	✓			✓
B-17	60	✓			✓
B-18	80	✓	✓	✓	
B-19	60	✓			✓
B-20	60	✓			
B-21	100	✓			
TOTAL	~1600	~21	~7	~7	~9

Notes:
[(1)] Subject to change based on site conditions
[(2)] SPT: Standard Penetration Test
[(3)] CPT: Cone Penetrometer Test
[(4)] VEL DH: Downhole velocity (VEL) test
[(5)] VEL PS Log: PS Suspension log velocity (VEL) test

3.1.2 Laboratory Testing Program

A laboratory testing program is performed on soil and rock samples collected from the site investigation program in accordance with the regulatory guidance of RG 1.138 (Reference 8.65) to obtain data for the analysis and design of the BWRX-300 RB. The scope and extent of the BWRX-300 laboratory testing program address the specific requirements of deeply embedded BWRX-300 design that requires a reliable set of data from laboratory tests for developing geotechnical inputs characterizing the properties of each subgrade material present at the site.

A laboratory testing program is implemented that depends on the site-specific subsurface conditions, the specific analysis requirements, and the need for sufficient data to adequately characterize variations in subsurface material properties. A sufficient number of laboratory tests are performed to minimize the uncertainties in the design related to these geotechnical input

parameters by providing reliable estimates for the statistical parameters (mean and standard deviation values) of the measured material properties. The systematic (bias) errors are minimized by a carefully executed equipment calibration and sample management programs. Estimates of measuring bias are developed based on comparisons of measurements of physical parameters obtained from different types of subsurface material property tests.

Testing to estimate strength parameters for appropriate rock discontinuities in bedrock units should be completed using appropriate methods that may include direct shear test (References 8.66, 8.67), triaxial strength tests (Reference 8.68), and appropriate methods identified in RG 1.138 (Reference 8.65). This testing shall determine the strength parameters (e.g., peak friction angle, residual friction angle, and apparent cohesion) of discontinuities and similar weak planes in rock. Testing of artificial interfaces may be completed to determine the strength properties at the interfaces with the RB structures.

At a minimum, the laboratory tests of soil materials include:

- Index testing (classification, weight, plasticity, grain size)
- Strength testing (shear tests, triaxial tests)
- Deformability tests (triaxial tests, consolidation tests)
- Permeability
- Chemical testing (chlorides, sulfates, pH, Resistivity)
- Dynamic tests (Resonant Column Torsional Shear (RCTS), cyclic triaxial)

The minimum laboratory tests required to develop properties for rock materials include:

- Uniaxial Compressive (UC) strength,
- Triaxial compressive strength and elastic moduli,
- Direct shear tests,
- Petrography,
- Dynamic tests (sonic pulse wave velocity, Free-Free Resonant Column velocity tests)

Other tests, such as the expansion, creep, mineralogy, erodibility, durability, X-ray diffraction tests may be performed on an as-needed basis.

3.1.3 Characterization of Rock Mass Properties

The properties of rock are characterized based on the information collected from the site investigation and laboratory testing programs described in Sections 3.1.1 and 3.1.2. Rock joints, bedding planes, discontinuities fracture and other weak zones are evaluated to determine:

- the type of temporary excavation support and improvements required during construction;
- groundwater conditions and required seepage control measures; and
- possible effects on the rock pressure loads on the RB shaft.

The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones may affect methods used to excavate rock for construction of the shaft. Methods that are used to compensate for these weak zones include:

- over-excavation and backfilling;
- internal structural support;
- spot or pattern rock reinforcement (i.e., rock bolts or anchors); and
- surface treatments (i.e., mesh, straps, shotcrete).

Additionally, the existing groundwater conditions and the potential control of seepage through cavities, fracture zones, joints, bedding planes, and discontinuities is considered. Seepage control may include slurry walls, grouting prior to excavation, grouting during the excavation, freezing, drains, dewatering wells, sumps and other methods. The existing groundwater conditions and appropriate modifications to the rock mass classification, consistent with the selected method, shall be determined as part of the Site Investigation Program in Section 3.1.1.

The in-situ state of stress in the bedrock shall be evaluated. This process shall include reviewing the state of stress in the crust as part of evaluating the tectonic framework and unrelieved stresses in bedrock near the site. A review of regional and/or local references that evaluate the current state of stress in the crust and the potential for horizontal stresses from tectonic activity, residual strains, or topographic conditions shall be used to assess the likelihood for increased horizontal stress in the bedrock. Based on the results of this review, in-situ tests like those shown in Table 3-1 may be considered to make site-specific measurements of the in-situ state of stress in bedrock formations as part of the geotechnical borings and borehole televiewer tests. All potential and/or appropriate tests for measuring in-situ stress are not identified in this document because the appropriate tests will be specific to each site.

Discontinuities and other zones of weakness within the rock mass may also control the stability of individual blocks or the rock mass when the orientation is disadvantageous and/or the spacing of discontinuities is sufficiently dense. The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. These discontinuities or weak zones may form a system of blocks or wedges where strength within the individual blocks is high, but strength along the weak zones between the blocks is highly anisotropic.

To adequately assess and consider weak zones in rock masses, RG 1.132 (Reference 8.64) and NUREG/CR-5738 (Reference 8.2) provide guidance on geologic mapping, logging and characterizing of rock materials. Geologic mapping and geotechnical borings described in Section 3.1.1 are used to characterize the intact rock, rock discontinuities, and the rock mass. Frequently, optical and acoustic televiewers (OTV/ATV) are used in conjunction with geologic mapping and oriented or classical rock coring methods to map the depths, orientations, aperture, and other characteristics of the discontinuities. The type of information and testing required for the rock mass will depend on the specific subgrade conditions as well as the rock mass classification selected for the site. When other data or geologic mapping indicates near vertical discontinuities may be present, inclined borings may be used to properly characterize the orientation and strength of near vertical discontinuities.

Empirical engineering and geo-mechanical rock mass classifications, such as the Rock Quality Designation (RQD) index, the Rock Tunneling Quality (Q) index, the 1976 and 1989 versions of the Rock Mass Rating (RMR) system, and the Geologic Strength Index (GSI), are used to quantitatively characterize the geologic and engineering parameters of rock masses (FHWA, 2009). These classifications often consider a variety of parameter ratings that are assigned based on the observations and measurements from characterized rock mass and may incorporate the proposed excavation techniques. Frequently, a range of parameter ratings are considered because a range of rock mass characteristics are encountered during subsurface characterization and multiple classifications systems may be considered to incorporate uncertainty in the parameter estimates.

Estimates of RQD may be made following NUREG/CR-5738 (Reference 8.2) on recovered rock cores and confirmed using OTV/ATV data or estimated from mapped or scanned surfaces based on the average number of discontinuities or volumetric joint count (Hoek et al. 2013, Reference 8.10).

RMR may be estimated following the parameters and ratings established by Bieniawski (1976, 1989, Reference 8.11). In order to use the RMR system, a rock mass is divided into different structural units defined by changes in rock type or major changes within a rock type, such as faults, fracture zones, or the spacing of discontinuities that may cause a change in the rock mass behavior. The RMR then considers semi-quantitative parameters for each structural region, which include the strength of the intact rock, RQD, the spacing of discontinuities, the condition of the discontinuities, the groundwater conditions, and the orientation of the discontinuities. Even though GSI is now commonly used directly without an estimate based on RMR, RMR is retained because previous studies have indicated better estimates using RMR for the rock mass deformation modulus of moderate to strong rock masses (Galera et al., 2007, Reference 8.12).

GSI may be estimated using qualitative charts relating the structure of the rock to the surface condition of joints for different types of rock masses (e.g., Hoek and Brown, 2018, Reference 8.13). Originally, the GSI system was developed for rock masses where block sliding and rotation was the primary means of failure without failure of the intact rock blocks, but has been extended to additional charts for other types of rock masses and geologic environments (Hoek and Brown, 2018, Reference 8.13). An appropriate GSI chart must be selected for the project site.

GSI may also be estimated semi-quantitatively for rock masses where block sliding, and rotation is the primary means of failure. This semi-quantitative method was developed for use when a qualified and experienced geologist or engineering geologist does not observe the rock mass and is recommended to supplement and not replace the qualitative estimates by a qualified and experienced professional. The quantitative input includes the RQD and the joint condition ($JCond_{89}$). Similar to the GSI, the $JCond_{89}$ value is based on a qualitative evaluation of the discontinuity surface and other features, including persistence, aperture, roughness, infilling, and weathering (Hoek et al., 2013, Reference 8.14). Alternatively, the $JCond_{89}$ may be estimated from a reduced set of estimates known as the joint roughness number (Jr) and joint alteration number (Ja) following Hoek et al. (2013, Reference 8.14). The semi-quantitative relationships for GSI and $JCond_{89}$ from Hoek et al. (2013) are provided below:

$$\mathrm{GSI} = 1.5\mathrm{JCond}_{89} + \frac{\mathrm{RQD}}{2} \tag{3-1}$$

where: $$\mathrm{JCond}_{89} = 35\frac{\left(\frac{\mathrm{Jr}}{\mathrm{Ja}}\right)}{\left(1+\frac{\mathrm{Jr}}{\mathrm{Ja}}\right)}$$

As described in RG 1.132, characterization of the shear strength for planar discontinuities, such as bedding planes, faults, fracture zones, joints, and shear zones typically include laboratory testing of subsurface discontinuities recovered from samples (e.g., direct shear and triaxial compressive strength tests) or, less commonly, in-situ tests of the discontinuities under specific loading conditions. Because the most common method is testing recovered subsurface samples, empirical corrections are required for surface roughness, intact surface strength, and the scale of the tested sample (e.g., Barton-Bandis criterion).

When the rock discontinuities are filled with another material, the shear strength may decrease or increase depending on the type of infill material. Testing of the infill material is required when there is a significant thickness of weaker material that may control the strength of the discontinuity. When a nonlinear relationship between shear strength and normal stress (e.g., Barton-Bandis criterion) is not desired, the equivalent friction angle and cohesion may be determined from the tangent to the nonlinear relationship for the shear strength of planar discontinuities.

Cavities in the rock mass from karst or dissolution may decrease the effective rock mass modulus and create a highly variable interface between the rock and overburden. The presence of cavities should be identified during the subsurface investigation. Consistent with RG 1.132, the spacing and depth of investigation locations should be reduced to detect the anticipated features.

A grouting program may be required to fill cavities and control seepage. The grouting program should include the potential to remove infilling from cavities using a water wash and fill the cavities as much as possible with grout. Replacing infill or open cavities with grout should increase and control variations in the rock mass modulus around and beneath the structures. Contact grouting is also required after construction of the shaft to avoid irregular external loading from voids – natural or due to overbreak during construction – on the exterior of the shaft. The rock surface may require modification through excavation or ground improvement to avoid significantly different stiffness along the shaft. Epikarst may form pinnacles or similar features that may result in variable stiffness along the shaft near the bedrock and overburden interface. The effect of potential cavities in the rock mass and variations at the bedrock and overburden interface on shaft deformation are evaluated on a site-by-site basis.

3.2 Construction Inspection and Testing Program

3.2.1 Excavation and Foundation Inspections and Testing

Excavation and foundation inspections and testing programs are implemented for the BWRX-300 that meet the geotechnical and foundation requirements of the NRC Inspection Manual 88131 (Reference 8.15), including:

A. Key Site Parameters are verified by checking if the required values for average allowable static bearing capacity and maximum allowable dynamic bearing capacity for normal plus SSE loading have been met at the excavation depth.

B. Soundness of the exposed rock is checked by qualified personnel to confirm the results of rock mass characterization described in Section 3.1.3. This includes visual inspection and testing of:

- Rock material properties, such as rock type, color, particle size, hardness, and strength.
- Rock mass properties, such as rock structure, shear strength, deformation modulus, hydraulic conductivity, and attitude.

3.2.2 Building Structure Construction Inspections and Testing

The BWRX-300 RB construction inspection and testing program satisfy the structural concrete activities requirements of the NRC Inspection Manual 88132 (Reference 8.16) and structural welding inspection requirements of NRC Inspection Manual 55100 (Reference 8.17). The program includes:

- The visual surface inspection acceptance criteria that include quantitative limits for the appearance of leaching or chemical attack, pop outs or surface voids, scaling, spalling, corrosion staining, settlements, and cracks.
- ACI 349.3R guidance (Reference 8.18), which is recommended by ASME XI Rules for Inservice Inspection of NPP Components, Subsection IWL for visual inspections of exposed surfaces. ACI 349.3R requires that accessible concrete surfaces do not have voids greater than 2 inches; scaling is limited to 8 inches in diameter and 0.75 inches in depth; and cracks are limited to widths of 0.04 inches or smaller.
- ASME XI, Subsection IWL 1220 (b) and (d) exempts concrete surfaces that are covered by a liner or adjacent to a foundation or backfill from detailed visual inspections.
- Concrete surfaces exposed to soil, backfill, or groundwater are evaluated to determine susceptibility of the concrete to deterioration and the ability to perform the intended design function under conditions anticipated until the structure no longer is required to fulfil its intended design function. The evaluation includes the following:

  a) Existing subgrade conditions, including groundwater presence, chemistry, and dynamics; aggressive below-grade environment, or other plant-specific conditions that could cause accelerated aging and degradation.

  b) Existing or potential concrete degradation mechanisms, including, but not limited to, aggressive chemical attack, erosion and cavitation, corrosion of embedded steel, freeze-thaw, settlement, leaching of calcium hydroxide, reaction with aggregates, increase in permeability or porosity, and combined effects.

  c) Design and construction criteria associated with the inaccessible concrete, including structural design, detail and reinforcement, design recommendations implemented with regard to environmental exposure conditions, materials used, mixture proportioning, concrete production and placement, design and construction codes used, conformance of the structure to original design and performance of any reanalysis.

d) Condition of installed protective barrier systems, such as membranes, coatings, grout curtains, special drainage systems, and dewatering systems

e) Any condition-monitoring programs being implemented, such as settlement monitoring, groundwater monitoring, condition surveys, and non-destructive examinations

f) Requirement for the examination of representative samples of below-grade concrete, when an aggressive below-grade environment is present

g) Based upon the evaluation of (b) above, it is necessary to document the condition monitoring program, including required examinations and frequencies, to be implemented for the management of deterioration and aging effects of the subgrade concrete surface. This program shall be incorporated into the plans and schedules required by IWA-1400(c) and IWA-6211(a).

- Present sufficient design details for the concrete mudmat and any excavation retaining structure, including constructing a test section for staff observation.
- Demonstrate adequate installation of the waterproofing system and achievement of minimum required coefficient of friction for the waterproofing system of the CB, TB and RwB foundations.
- Demonstrate key dimensions and volumes have been achieved, such as wall thickness tolerances based on credited shear walls in seismic analysis.

The construction inspection and testing program covers the project phase up through plant commissioning. The program demonstrates that the facility is constructed to the requirements in the design drawings and specifications and provides a baseline of data for the continued In-Service Monitoring Program, described in Section 3.3.

3.2.2.1 Concrete Compressive Strength Testing Frequency

A compressive strength testing program is performed on concrete samples collected from safety-related concrete to ensure that the concrete placed during construction meets design specifications. Following the guidance of RG 1.142, this in-process concrete strength testing program is performed in accordance with Section 5.6.2.1 of ACI 349-13 (Reference 8.24) with certain exceptions as noted in RG 1.142. An additional sampling frequency requirement is adopted for the BWRX-300 that is beyond current regulatory guidance to address the specific concrete testing requirements for construction of smaller SMRs identified in NUREG/CR-7193, Section 1.5.6 (Reference 8.1).

The testing frequency affects the number of data points used to calculate the predictability of concrete strength, including the standard deviation of concrete mixes to ensure adherence to design specifications and Quality Assurance/Quality Control (QA/QC) requirements. Section R5.6.2 of ACI 349-13 (Reference 8.24) Commentary states that testing frequency does not have much effect on accuracy of standard deviation calculation after 25 or more tests are performed on one mixture of a given class of concrete. Based on this finding, ACI 349-13, Section 5.6.2.1 permits less frequent concrete compressive strength tests if 30 or more consecutive tests have been performed.

The volume of safety-related concrete placed during the construction of the BWRX-300 is an order of magnitude smaller than the volume of safety-concrete used for the construction of a typical large LWR power plant. Due to the smaller concrete volumes associated with the BWRX-300 construction, the sampling frequency requirements of ACI 349-13 (Reference 8.24), Section 5.6.2.1, may not result in a total of 30 or more tests for concrete mixes for various BWRX-300 concrete components. Therefore, an additional sampling requirement is adopted for the BWRX-300 to ensure that a statistically significant sample size is attained for each group of concrete components based on final concrete mix requirements. To ensure at least 30 tests are performed for each group of concrete components, an additional requirement for concrete testing frequency is applied as follows:

- The final as-designed concrete components (e.g. RB basemat and RB shaft walls/floors) are sorted based on concrete mix design requirements;
- The volume of each group of components, V_i (yd^3), is determined; and
- An additional volumetric sampling requirement is imposed, such that at least one test is performed for every $(V_i/30)$ yd^3 of concrete placed.

For each group of BWRX-300 safety-related concrete placed each day, the testing frequency requirements are summarized as follows:

- If less than 30 consecutive tests have been performed, the minimum testing frequency is determined based on the maximum of:
  - Once per day;
  - One test per one 30th of the total volume placed ($V_i/30$ yd^3); and
  - One test per 100 yd^3 of concrete placed.
- If 30 or more consecutive tests have been performed and the standard deviation of those tests (σ) is less than 600 psi, the minimum test frequency is the maximum of:
  - One test for each production shift;
  - One test per $\left(100yd^3 + 50yd^3\left(\frac{600psi-\sigma}{100psi}\right)\right)$ of concrete placed; and
  - One test per 200 yd^3 of concrete placed.

Regardless of test frequency, any concrete mixes that do not meet design specifications and/or QA/QC requirements are rejected and remediation measures taken to address the quality deficiency.

3.3 In-Service Monitoring Program

3.3.1 Scope of Structures Monitoring and Aging Management Program

The scope of the BWRX-300 Structures Monitoring and Aging Management Program (SMAMP) is the in-service condition monitoring of structures and management of aging effects, i.e. structural degradations and deformations. The program begins upon the successful commissioning of the plant and terminates upon the completion of plant decommissioning. Inspections and monitoring

during construction and commissioning are described in Section 3.2. The BWRX-300 also implements post-construction testing and in-service surveillance programs for below-grade structural members and foundations, such as periodic examination of inaccessible areas, monitoring of groundwater chemistry, and monitoring of settlements and differential displacements. The data obtained from monitoring of settlements and differential displacements that together with results of numerical sensitivity evaluations described in Section 4.3.4.5, can be used to detect and analyze potential instabilities in the subgrade surrounding the deeply embedded BWRX-300 RB.

The purpose of the in-service monitoring programs is to monitor the condition of BWRX-300 structures over their design lives to ensure that credited safety functions are maintained as well as overall structural integrity. The overall integrity of all structures, regardless of safety classification, is important, so that plant personnel can safely maintain plant facilities during service and through decommissioning. The purpose of the programs is achieved by providing a framework for the timely identification and management of structural degradation mechanisms.

3.3.2 Framework of Structures Monitoring and Aging Management Program

The framework of the BWRX-300 SMAMP is based on the three-tier evaluation hierarchy of ACI 349.3R (Reference 8.18), which is shown schematically on Figure 3-2. The evaluation of structures begins upon successful commissioning of the plant and continues at prescribed intervals until the decommissioning of the plant is complete. The inspection intervals are defined in the BWRX-300 SMAMP following the guidance in Chapter 6 of ACI 349.3R (Reference 8.18). Individual utilities may elect to perform supplemental walkdowns in between SMAMP-prescribed inspection intervals using plant maintenance personnel; however, the personnel performing evaluation walkdowns under the SMAMP shall be qualified per the requirements in Chapter 7 of ACI 349.3R (Reference 8.18).

The framework of the SMAMP depends on the identification of applicable structural degradation mechanisms and setting criteria to determine appropriate action whenever these degradations are observed. As shown on Figure 3-2, the SMAMP framework has the following three tiers:

- Tier 1: Evaluation of Structure Against First-Tier Criteria

  First-Tier Criteria provide qualitative and quantitative thresholds for visual inspection or condition surveys that, if met, the structural condition deemed acceptable without further evaluation in accordance with Section 5.1 of ACI 349.3R (Reference 8.18).

- Tier 2: Evaluation of Structure Against Second-Tier Criteria

  Second-Tier Criteria provide qualitative and quantitative thresholds for observed degradation conditions that are determined to be inactive after review. In accordance with Section 5.2 of ACI 349.3R (Reference 8.18), in determining if an observed degradation is inactive, a comparison of current observed conditions with the results of prior inspections may be required. Observed conditions exceeding Second-Tier Criteria proceeds to a Tier 3 evaluation.

- Tier 3: Evaluation of Structure After Enhanced Inspections, Testing, and Analysis

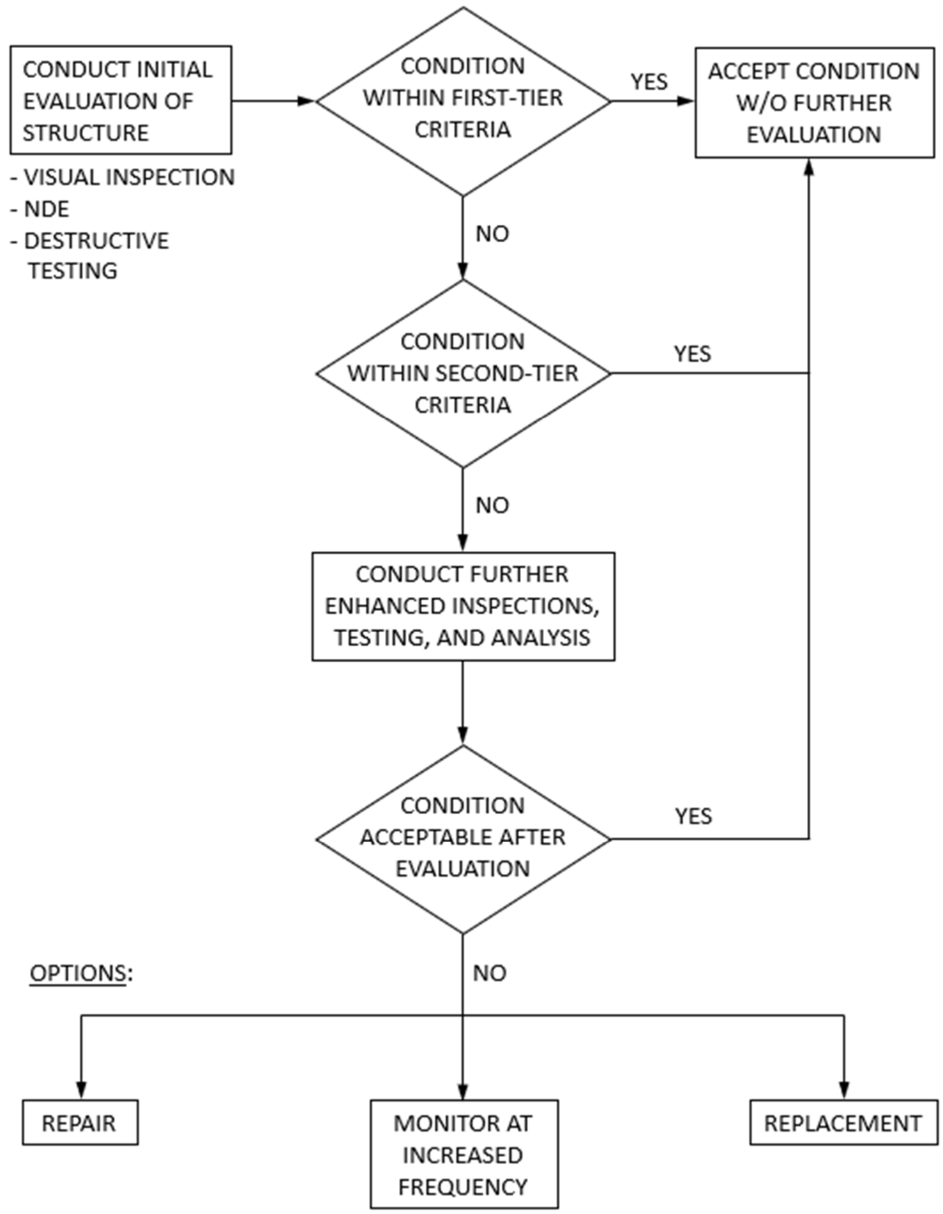
  Tier 3 evaluations involve more enhanced methods to obtain information related to structural condition and function. These enhanced methods may include nondestructive testing, destructive testing, and/or re-analysis of structural capacity and behavior under degraded conditions. A summary of evaluation techniques is provided in Section 3.5 of ACI 349.3R (Reference 8.18). If the result of the Tier 3 evaluation is a corrective action, e.g., repair modification, then the corrective action will follow the guidance found in Chapter 8 of ACI 349.3R (Reference 8.18).

Evaluations under the SMAMP will follow the decision logic shown on Figure 3-2 until the condition of evaluated structures is found to be acceptable or corrective actions are taken to bring the evaluated structures back into an acceptable condition. Corrective actions may include repair modifications, increased monitoring frequencies, or replacement of defective structural components. Table 3-3 provides examples of degradation conditions and corresponding SMAMP criteria for accessible concrete structures, while Table 3-4 provides the same for accessible steel structures. Additional criteria may be developed for site-specific structures and design features.

The SMAMP will also include the periodic sampling and testing of groundwater and the need to assess the effect of any changes in its chemistry on below-grade concrete structures. It is important to determine, whether below-grade concrete structures are exposed to an aggressive environment. This can be accomplished through testing soil or groundwater adjacent to these structures for pH, chloride concentration, and sulfate concentration. Per Section XI.S6 of NUREG-1801 (Reference 8.19), an aggressive environment in soil or groundwater exists under the following conditions:

- pH < 5.5
- chlorides > 500 ppm
- sulfates > 1,500 ppm

If protective coatings are relied upon to manage the effects of aging for any structures, the SMAMP will address protective coating monitoring and maintenance according to the guidance provided in Sections 5.1.4 and 5.2.4 of ACI 349.3R (Reference 8.18).



Note: Reproduced based on Fig. 5.1 of ACI 349.3R-02

Figure 3-2: Framework of Structures Monitoring and Aging Management Program

Table 3-3: Degradation Conditions and Criteria for Accessible Concrete Structures

Degradation Condition	First-Tier Criteria	Second-Tier Criteria
Leaching, Efflorescence, and/or Chemical Attack	Absence of condition(1)	Condition present(3)
Abrasion, Erosion, and/or Cavitation	Absence of condition(1)	Condition present(3)
Drummy Areas	Absence of condition(1)	Condition present with depth > cover concrete thickness(3)
Popouts and/or Voids	< 20 mm (3/4 in.) in diameter or equivalent surface area(1)	< 50 mm (2 in.) in diameter or equivalent surface area(3)
Scaling	< 5 mm (3/16 in.) in depth(1)	< 30 mm (1-1/8 in.) in depth(3)
Spalling	< 10 mm (3/8 in.) in depth and 100 mm (4 in.) in any dimension(1)	< 20 mm (3/4 in.) in depth and 200 mm (8 in.) in any dimension(3)
Signs of steel reinforcement or anchorage component corrosion	Absence of condition(1)	Condition present(3)
Deflections, settlements, or other physical movements	Absence of condition(1)	Passive settlements or deflections within original design limits(3)
Leakage/Seepage (presence of water)	Absence of condition(2)	Condition present, but within original design limits of active leak-detection system and the leaking material and source do not present any adverse consequences(4) (5)
Cracking (new)	Passive cracks < 0.4 mm (0.015 in.) in maximum width(1)	Passive cracks < 1 mm (0.04 in.) in maximum width(3)
Crack Growth (i.e. active cracking)	Absence of condition(1)	Absence of condition(1) (3)

(1) Section 5.1.1 of ACI 349.3R (Reference 8.18)

(2) Section 5.1.2 of ACI 349.3R (Reference 8.18)

(3) Section 5.2.1 of ACI 349.3R (Reference 8.18)

(4) Section 5.2.2 of ACI 349.3R (Reference 8.18)

(5) Section XI.S6 of NUREG-1801 (Reference 8.19)

Table 3-4: Degradation Conditions and Criteria for Accessible Steel Structures

Degradation Condition	First-Tier Criteria	Second-Tier Criteria
Corrosion and/or corrosion stains	Absence of condition[(1) (2)]	Condition present, but determined acceptable after further review[(3) (4) (5)]
Bulges or depressions in liner plate	Absence of condition[(1)]	Condition present, but determined acceptable after further review[(3)]
Cracking/degradation of base or weld metal	Absence of condition[(1)]	Condition present, but determined acceptable after further review[(3)]
Leakage/Seepage (presence of water)	Absence of condition[(1)]	Condition present, but within original design limits of active leak-detection system and the leaking material and source do not present any adverse consequences[(3)]
Detached embedments or loose bolts	Absence of condition[(2)]	Condition present, but determined acceptable after further review [(4)]

(1) Section 5.1.2 of ACI 349.3R (Reference 8.18)

(2) Section 5.1.3 of ACI 349.3R (Reference 8.18)

(3) Section 5.2.2 of ACI 349.3R (Reference 8.18)

(4) Section 5.2.3 of ACI 349.3R (Reference 8.18)

(5) Section IWE-3500 of ASME XI (Reference 8.20) provides a threshold of 10% loss of nominal wall thickness.

3.4 Field Instrumentation Plan

Field instrumentation that beyond the current regulatory guidelines, is deployed to monitor the magnitude and distribution of pore pressure and amount of deformation during excavation, construction, loading and continuing through the BWRX-300 plant operation. The instrumentation provides recordings that can frequently be benchmarked against design estimates. Short-term and long-term settlement monitoring plans are developed that can detect both vertical and horizontal movements in and around the structures, as well as differential distortion across the foundation footprint and differential settlements between the CB, TB, RwB and RB foundations.

The specific locations of the sensors inside and outside of the RB shaft are dictated by the subsurface conditions and areas identified in the design where maximum stress, strain, and pore pressures are anticipated along the perimeter of the shaft. The definitive number of instruments is established during design stages of the monitoring system considering that the field instrumentation system shall be capable of:

A. Measuring the rate of heave during excavation, especially at the end of excavation and at the bottom center and edges of the shaft.

B. Measuring the rate of lateral displacement of excavation walls, throughout its depth, during and at end of excavation.

C. Measuring the distribution of pore pressures around and below the RB shaft.

D. Measuring the total settlement and tilt of the RB shaft, during construction, loading, and operation; this will require deploying a system of sensors and survey monuments throughout the perimeter of the shaft at bottom, medium depth, and plant grade.

E. Measuring settlement of the auxiliary and surrounding structures of the BWRX-300.

Figure 3-3 indicates the required implementation period that the field instrumentation has to accommodate. Some instruments will be temporary while others are permanent. Some instruments, such as piezometers, are installed prior to excavation. Installation for extensometers or other survey monuments are to be taken at the appropriate stage of the BWRX-300 life.

To achieve the required monitoring capabilities, the field instrumentation consists of four primary elements:

1. Piezometers to measure pore pressure distribution. Vibrating wire piezometers are preferred for this purpose as they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis. Piezometers should be screened at elevations that are representative of the site-specific hydrogeologic conditions.

2. Settlement monuments placed directly on concrete, preferably on the corners of the structures at grade that are accessible with conventional surveying equipment.

3. Settlement sensors and extensometers used for settlement prone soils or deformation prone rock masses.

4. Earth pressure sensors to monitor vertical and lateral pressure along the walls of the shaft.

For deployment in soft soil conditions, settlement sensors are installed within a borehole attached by a Borros anchor as described in Reference 8.21. For hard soil and rock conditions, sensors may consist of rod type extensometers anchored below loading points. The borehole extensometer includes anchors, extension rods and a reference head. The anchor is connected to the head of the instrument by extension rods typically placed within a protective sleeve. This sleeve ensures that the rods can move freely and translate all movement of the anchor to the tip of the rod. The movement of the rock or soil mass relative to the head can then be calculated by measuring the displacement of the tip of the extension rod to a reference plate located in the head of the extensometer as the one described in Reference 8.22. The instrument can be used to measure deformation of laterally loaded retention walls and to monitor settlement in foundations.

The groundwater levels at the site are monitored using pressure transducers installed in multiple screened wells installed across the site. This data provides groundwater elevations, groundwater flow direction(s) and groundwater gradients. This information is used during excavation and construction for estimating seepage rate, short-term dewatering rates, and effective stresses under static and dynamic conditions.

When practical and applicable, sensors are connected to a datalogger(s) programmed to read the sensors periodically. Some of these sensors are installed in cased boreholes and the sensors can be removed, maintained, or replaced during the needed phases of the project. Other sensors, such as the earth pressure sensor, need to be buried in the subsurface and cannot be removed or replaced once backfilled. Such sensors are installed with redundancy to monitor the necessary data for the specific duration of the project phase when such data is used.

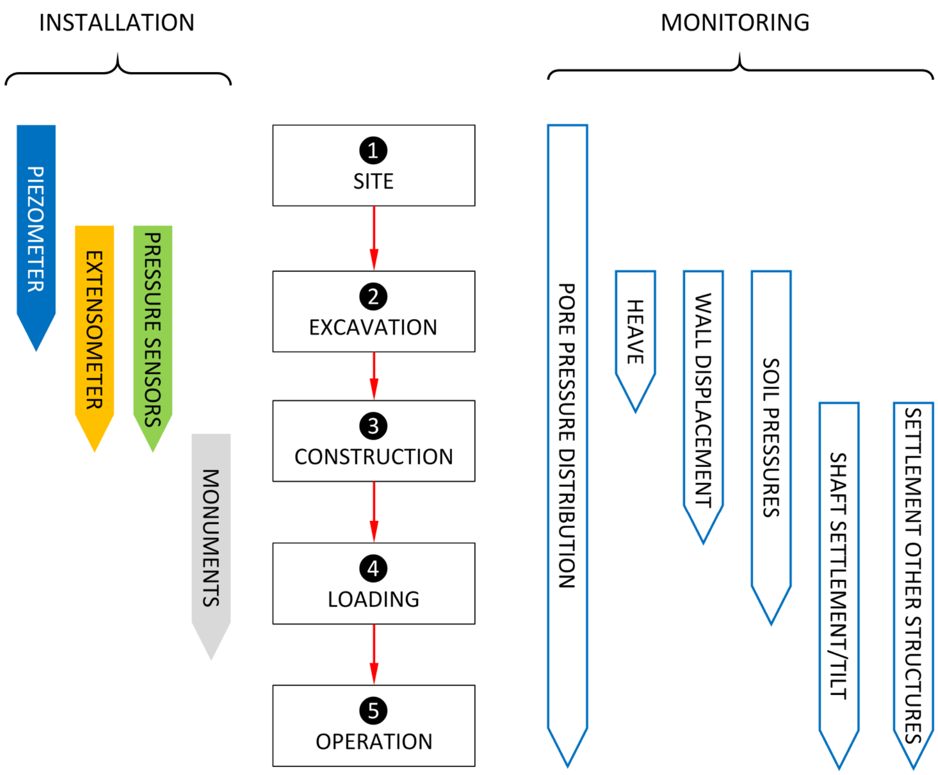


Figure 3-3: Field Instrumentation and Monitoring

3.5 Summary of Investigations, Testing, Inspection and Monitoring Programs

The following BWRX-300 requirements and innovative approaches that are related to the site investigations, testing, inspection, and monitoring programs, presented in this section of the report, may be referenced during future licensing activities:

(1) The site investigation program requirements provided in Section 3.1.1 include recommendations for the minimum number of borings and field tests, types of field tests, boring locations and depths that are beyond the current guidance of RG 1.132 (Reference 8.64).

(2) The laboratory testing program approach presented in Section 3.1.2 includes the minimum requirements for laboratory testing of soil and rock materials that are beyond the current guidance of RG 1.138 (Reference 8.65).

(3) The approach for characterization of rock mass properties presented in Section 3.1.3 include examples of empirical engineering and geomechanical rock mass classifications to quantitatively characterize the geologic and engineering parameters of rock masses and recommendations for characterization of the shear strength of planar discontinuities that are beyond the current guidance of RG 1.132 and RG 1.138.

(4) The construction inspection and testing program approach presented in Section 3.2, include requirements for the minimum frequency of concrete compressive strength tests,

described in Section 3.2.2.1, that are beyond the current guidance of RG 1.142 when applied to SMRs.

(5) The in-service monitoring approach, presented in Section 3.3 include surveillance programs for below-grade structural members and foundations and monitoring of settlements and differential displacements that together with the results of numerical sensitivity evaluations can be used to detect and analyze potential instabilities, as described in Section 4.3.4.5, that are beyond the current guidance of RG 1.160 when applied to deeply embedded SMRs.

(6) The field instrumentation approach, described in Section 3.4 is used for monitoring and evaluating possible properties or instabilities changes of subgrade materials during the excavation, construction and operation of BWRX-300, which together with the results of FIA, described in Section 4.3.4, are beyond the current guidance of SRP 2.5.4.

4.0 FOUNDATION INTERFACE ANALYSIS

The purpose of a FIA is to ensure that BWRX-300 RB, CB, TB and RwB structures and supporting media, soil and/or rock, meet the stability requirements and regulatory guidance of SRP 2.5.4, with emphasis in Subsections 2.5.4.3, 2.5.4.5, 2.5.4.6, and 2.5.4.10. The construction plans, including possible ground improvements, excavation support and foundation interface design are evaluated based on the results of FIA at different life stages of the BWRX-300 using non-linear models that have the capability of sequencing construction and loading and reproducing the stress and deformation fields. The results of the FIA are also used for verification of the RB shaft design as described in Section 5.1.3.

The scope and extent of BWRX-300 FIA are beyond the current regulatory guidance and address specifics related to the design and construction of deeply embedded SMRs identified in NUREG/CR-7193, Sections 1.5.10 and 1.5.11 (Reference 8.1). The predicted foundation interface behavior is compared against physical observations from the monitoring programs described in Sections 3.2, 3.3, and 3.4 to:

- allow for confirmation of the analyzed stability conditions;
- assess the effects of excavation and construction on the properties of in-situ subgrade materials;
- evaluate the effects of new loads or changes in the site conditions that may occur during the operation life of the BWRX-300 plant; and
- evaluate potential subsurface deformations that may originate from subsurface instabilities.

The implemented approach offers assurance that the actual observations are within expected ranges that have been anticipated during the design and prior to the construction.

4.1 Foundation Interface Analysis Model

The BWRX-300 stability is monitored throughout the remainder of its life stages (excavation, construction, loading, and operation) by implementing a benchmark process that provides a link between the expected and measured response of the system. The process involves development of a numerical model that examines the response that the BWRX-300 and its surrounding media exhibits due to alterations of in-situ subgrade conditions. Such responses are monitored, both through FIA model response and field measurements. The numerical model is calibrated using the field measurements to predict future response of the structure. This process represents a tool to reassure structural and site responses stay within the design bounds.

The FIA numerical model has the following features:

A. Three dimensional

B. Capability to incorporate non-linearity in the stress-strain behavior of soil and rock; this feature addresses the non-linear behavior at low and high strain, and even cases where physical instabilities may be present. The non-linearities in stress-strain of soils and rocks is captured by the use of constitutive models described in Section 4.2 that best fit the subgrade materials of the deployment site. These constitutive models range from the simplest elastoplastic

Mohr-Coulomb to other more sophisticated cases that incorporate strain-hardening/softening, strain dependent elastic and shear moduli, or rock failure criteria such as Hoek-Brown.

C. Interface modeling described in Section 4.3.1; allows the introduction of the response and failure criteria between geometric zones; this feature is necessary to analyze faults, rock slip surfaces, or other discontinuities around the structure. The interface modeling has non-linear modeling capabilities.

D. Interface modeling between soil/rock and structure described in Section 4.3.1; which is necessary to incorporate interaction between concrete and soil/rock via friction, accounting for the selected construction method and final configurations at the structure-soil/rock contacts. Non-linear behavior and separation are parts of the capability of this feature.

E. Structure modeling, which may be limited to the main civil/structural components of the RB: main walls, floors, pools, and auxiliary structures.

F. Soil/rock anchors and geogrids, which are used to simulate stabilization of the excavation and any associated potential failure surfaces.

G. Fluid-soil interaction, which may be considered if the modeling the position of a static, horizontal groundwater table is not sufficient for the complexities in the design and construction of the BWRX-300 RB. Pore pressures are dependent on the permeability of the subsurface media, the hydrogeologic configuration, and the dewatering strategies for construction and operation.

H. Staging analysis with time-dependent capabilities, which enables modeling the interaction of the structure and surrounding subgrade from excavation, through construction, loading and final operation. The model is capable of following stress/strain response as stress regimen changes from unloading during excavation to reloading after construction and during operation.

4.2 Subgrade Material Constitutive Models

Constitutive models define the relationship between the stresses and strains for different materials. Non-linear constitutive models are used for soils, rocks, and interfaces, or a combination of them.

The selection of the non-linear constitutive models for the BWRX-300 FIA is based on site-specific characteristics of the subsurface materials and the expected stress levels that result from dewatering, excavation, and loading. Regardless of the selected constitutive approach, the numerical model handles the potential for development of plastic zones or interfaces that can result from planes of weakness, presence of voids or cavities, or simple excess loading.

The parameters defining the soil and rock constitutive models are developed based on data obtained from the field and laboratory testing programs described in Section 3.1 and calibrated based on data collected from the field instrumentation program described in Section 3.4. This calibration includes modifying select input parameters for the soil and rock constitutive models or the interface models to better match the data collected from the field instrumentation program.

4.2.1 Soil Constitutive Models

Non-linear constitutive models are applied to soil materials. The Mohr-Coulomb failure criterion is typically used to represent shear failure in soil. Soils with Mohr-Coulomb behavior experience

purely linear, elastic deformation with increased stress. When the stress is large enough to bring failure upon the soil, the behavior turns fully plastic. The Mohr-Coulomb model is adequate for most soils although it is, in general, an oversimplification of the stress/strain behavior of many materials under significant loading. The key inputs required for defining Mohr-Coulomb elastoplastic constitutive models include the Young's modulus, Poisson's ratio, dilation angle, friction angle and cohesion intercept. The Mohr-Coulomb model is used for the BWRX-300 except if soils for a specific site cannot be modeled with a linear elastic behavior and the use of a single elastic modulus.

The Mohr-Coulomb constitutive model is used for the BWRX-300 FIA except if soils for a specific site cannot be modeled with a linear elastic behavior and the use of a single elastic modulus.

More advanced models are used for these site conditions, such as the strain-hardening/softening (HS) model. The HS model incorporates the effect that non-linear response has on the overall strength in soils as observed in laboratory specimens and field tests. This model eliminates the linear, elastoplastic simplification of Mohr-Coulomb, and it more closely models the behavior of soil. However, the HS model may be data intensive, as it requires a thorough and well-planned field work and laboratory testing program to be able to calibrate the model to site-specific data. For example, the Mohr-Coulomb constitutive model requires only the definition of an elastic modulus and shear strength parameters (cohesion, friction, and dilation angles). The advanced HS model requires additional stress-dependent input parameters to simulate the soil stiffness under different confining stresses and unloading and reloading stress conditions. As discussed in Section 5.1.4, the modeling refinements may offer little to no advantage from the Mohr-Coulomb constitutive modeling approach, if the confidence in the input data is not adequate.

4.2.2 Rock Constitutive Models

Discontinuities and other zones of weakness within the rock mass may control the stability of individual blocks or the rock mass when the orientation is disadvantageous and/or the spacing of discontinuities is sufficiently dense in weak rock. These discontinuities or weak zones may form a system of blocks or wedges where strength within the individual blocks is high, but strength along the weak zones between the blocks is highly anisotropic. Response to loading these jointed or fractured rock masses may involve complex interaction of compression, translation, wedging, rotation and, possibly, the generation of new joints from fracturing. The non-linear FIA can consider the rock mass as a discontinuous or continuous material depending on the size, density and configuration of the rock discontinuities and weak zones.

The Mohr-Coulomb failure criterion is typically used to represent shear failure in the rock. The Mohr-Coulomb failure criterion is sometimes also used in conjunction with the ubiquitous-joint model to include weak planes with specific orientations. The shear strength of rock mass and planar rock discontinuities are developed from the results of field investigations and laboratory tests as described in Section 3.1.3. The sampling and testing of rock units is carefully developed to test intact rock and the preferential weak planes.

The Generalized Hoek-Brown (GHB) model may be used to better represent the nonlinear stress-strain behavior of rock masses when Mohr-Coulomb constitutive models are not considered representative over a larger range of stresses. Specifically, the GHB model may better represent the response of an isotropic rock mass where the rock stiffness is nearly constant over a range of

stresses, but the shear strength is variable due to the presence of discontinuities and weak zones. The GHB model is applicable to rock masses with confining stresses below the transition to ductile failure, which is anticipated for most BWRX-300 deployment sites.

The GHB criterion uses intact rock measurements of UC strength with the GSI geomechanical rock mass classification, described in Section 3.1.3, to estimate adjusted strength and deformation parameters, Mohr-Coulomb friction angle and cohesion parameters, the uniaxial rock mass strength, and rock mass deformation modulus for the rock mass, under different geological conditions.

The following empirical equation developed by Hoek et al. (Reference 8.23) for the GHB failure criterion may be used:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{4-1}$$

where: σ_1 and σ_3 are the major and minor principal stresses, respectively;

σ_{ci} is the unconfined compressive strength developed based on of the intact rock based on regression analysis from multiple triaxial strength tests (Reference 8.23).

m_b, s and a are rock mass material constants.

The parameters m_b, s, and a that determine the shape of the nonlinear failure envelop are given by the following equations:

$$\begin{gathered} m_b = m_i exp\left[\frac{(GSI - 100)}{(28 - 14D)}\right] \\ s = exp\left[\frac{(GSI - 100)}{(9 - 3D)}\right] \\ a = \frac{1}{2} + \frac{1}{6}\left[e^{-GSI/15} - e^{-20/3}\right] \end{gathered} \tag{4-2}$$

where: m_i is obtained as a part of the regression analysis of the results of multiple triaxial strength tests used to determine σ_{ci} as described in (Reference 8.23)

s =1 and a = 0.5 for intact rock as noted in (Reference 8.23)

D is a disturbance factor that varies from 0 to 1 depending on the amount of disturbance to the rock mass from blast damage and stress relaxation.

Section 8 of Reference 8.23 provides a guidance for selecting a value of D. For the BWRX-300 deployment, a thinner layer of disturbed rock (e.g., $D = 0.5$) is anticipated for most sites to account for excavation damage and stress relaxation with undisturbed rock (e.g., $D = 0$) at some distance behind the face of the excavation.

4.3 Non-Linear Foundation Interface Analysis Approach

The FIA addresses the following aspects:

A. Interface modeling, described in Section 4.3.1, including both (a) contacts between structure and soil/rock, and (b) fault or joint planes or interfaces between bedding units in a geologic formation.

B. Structural modeling of the main civil/structural components of the BWRX-300 and auxiliary facilities, described in Section 4.3.2, along with varying live and dead loads throughout the construction process.

C. Fluid Soil Interaction, described in Section 4.3.3, to capture an adequate distribution of the space and time variation of pore pressures.

D. BWRX-300 life stages: siting, excavation, construction, loading, and operation described in Section 4.3.4.

4.3.1 Interface Models

4.3.1.1 Interfaces Between the Structures and the Subgrade Media

The behavior of the contact at the base might not be critical for the RB because sliding and overturning are likely controlled by the deep embedment. However, the behavior of contact between the walls and soil, influences the soil pressures exerted on the structure along its embedded depth. The contact behavior depends on the selected construction methodology and changes through construction. For example, the contact condition of the BWRX-300 RB outer wall, when poured using a slurry wall or rock face as formwork, is different than the contact gained from a typical construction and backfill/grouting process. Figure 4-1 provides a schematic showing interfaces between structure and the surrounding media.

The interface is modeled, as is the case for the soil, with the use of an elastoplastic relationship based on an elastic deformation modulus and shear resistance. Figure 4-2 shows an example of interface rheologic modeling typically used for BWRX-300 FIA. A series of spring couplers are simulated at the connecting grid points at the interface. Each spring is represented by an elastoplastic model with Mohr-Coulomb criterion for shear failure.

When interface elements are used to represent the structure and soil/rock interaction, node pairs are created at the interface. From a node pair, one node belongs to the structure and the other node belongs to the soil/rock. The relative displacements (i.e. slipping/gap opening) can be simulated through elastic-perfectly plastic springs between these two nodes. Typically, two sets of springs are used for interface elements. One elastic-perfectly plastic spring to model the gap displacement and one elastic-perfectly plastic spring to model slip displacement. The simulation of gaps opening between the structure and soil/rock can be achieved through activating a tension cut-off for the spring that does not allow any tension at the interface.

The parameters of the slipping spring can be taken from the material set of the adjacent soil/rock elements or strength tests on natural and artificial discontinuities from the site investigation, laboratory testing program and characterization programs as described in Sections 3.1.2 and 3.1.3. The development of the interface parameters should be consistent with the limitations and modeling guidance of the software and interface model used for the nonlinear FIA. A strength

reduction factor can be used to adjust the spring stiffness based on the roughness of interaction and soil/rock residual strength when the sliding occurs. It is also possible to assign strength properties to interface elements based on direct measurements. If planar geosynthetic products are used during construction of the wall, shear properties are assigned to the interface elements representative of shear properties at geosynthetic/soil interfaces.

As is the case for soil and rock material constitutive models, the use of complex modeling capabilities for modeling interfaces introduces the challenge of identifying adequate input physical parameters. To address the uncertainties in these input parameters, sensitivity analyses may be conducted by adjusting spring stiffness and shear strength directly or through strength reduction factors. These types of analyses provide insight to understand the uncertainty introduced by interfaces in the stress distribution and deformation response of the structure.

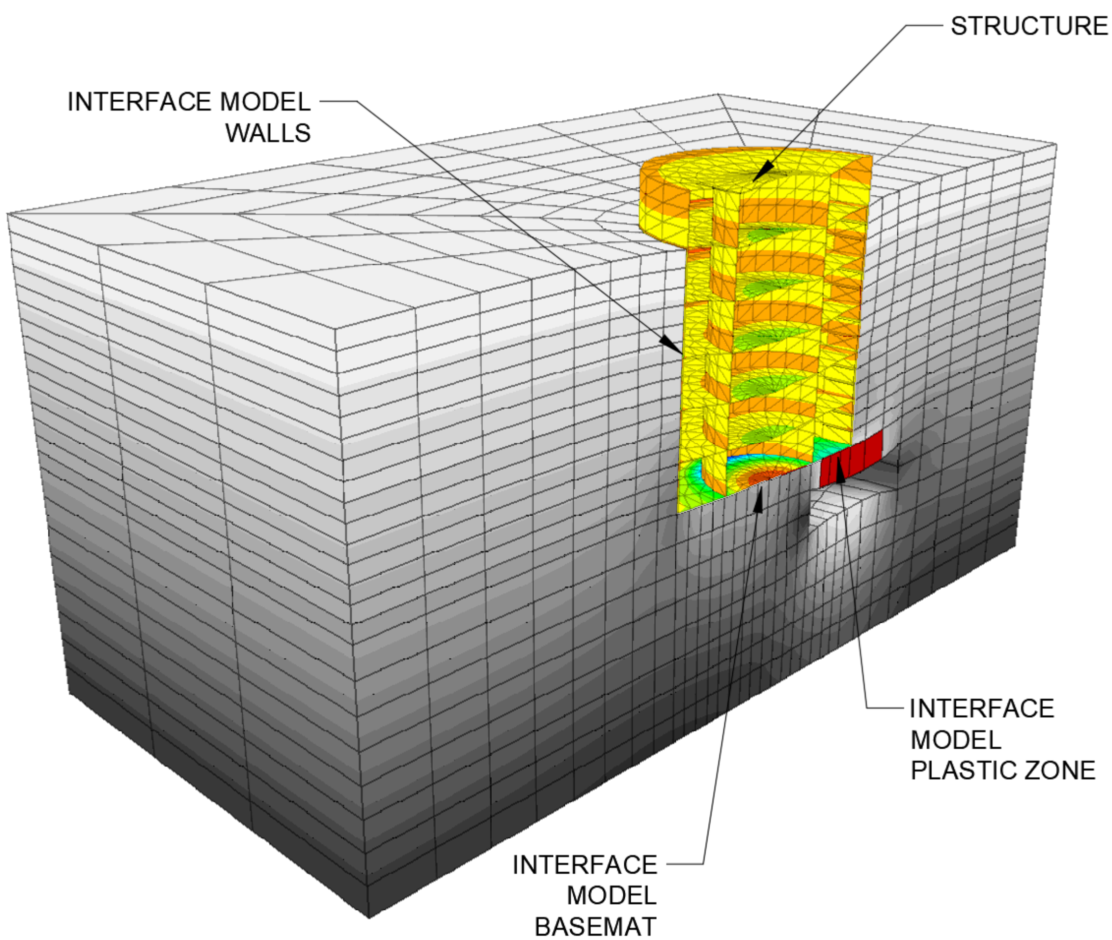


Figure 4-1: Location of Interfaces between Soil and Structure

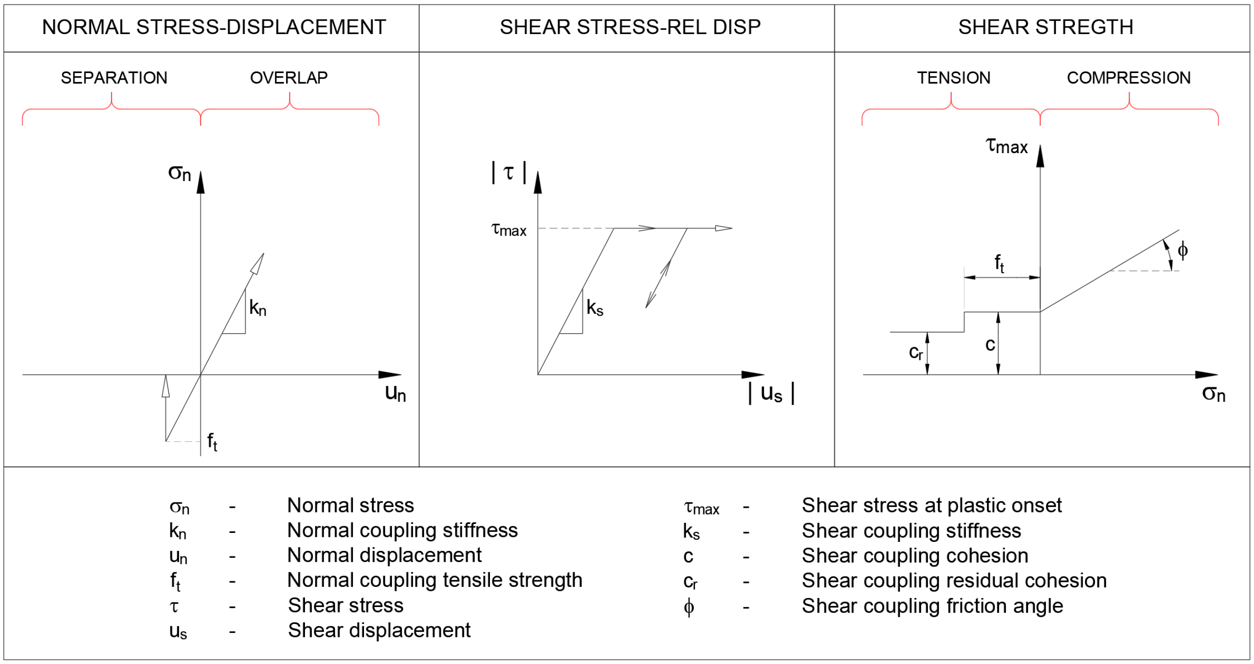


Figure 4-2: Interface Rheologic Modeling

4.3.1.2 Fault or Joint Planes or Interfaces Between Bedding Units in a Geologic Formation

The embedment depth of the BWRX-300 allows the possibility that soil rock interfaces, bedding interfaces, and other joints (Figure 4-3) may be in contact with the sides and base of the structure. These features may have planar or irregular configuration, and may be horizontal or with dipping, and even striking angles with respect to the position of the structure. The non-linearity and behavior of the joints are analyzed throughout the life stages of the reactor. These interfaces are modeled using similar interface modeling approaches as described in Section 4.3.1.1. The strength properties assigned to the interface elements along a rock discontinuity, i.e. bedding, are obtained from laboratory testing data described in Section 3.1.2. When multiple strength tests are performed for rock discontinuities, the weakest strength parameters can be used for the interface elements or sensitivity analyses may be completed similar to Section 4.3.1.1. Strength reduction factors may be used to adjust the spring stiffness and shear strength based on the strength of the interface where the sliding occurs.

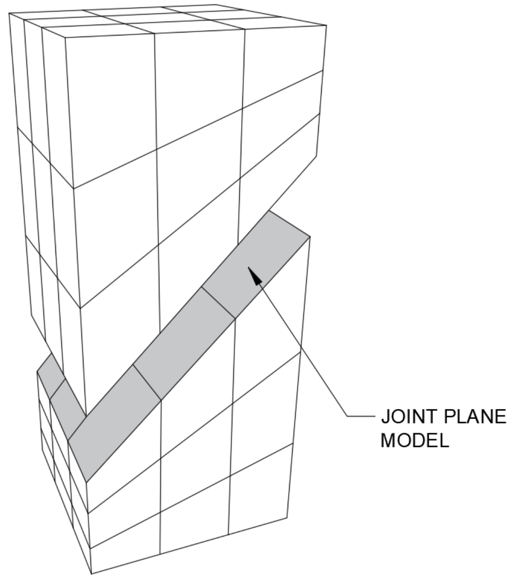


Figure 4-3: Joint Plane Model

4.3.2 Structural Elements Representation in the Foundation Interface Analysis Model

Elements are included in the non-linear models for the FIA models BWRX-300 to represent the RB structures, soil stabilization elements such as rock anchors, soldier piles, and stabilization walls and liners. Linear elastic material properties are assigned to the BWRX-300 structural members. The use of linear elastic properties for the structural members is adequate for capturing the interaction of the structure with subsurface materials because the subgrade materials or interfaces may undergo plastic behavior and experience large strain deformation quickly before the structure reaches the onset of inelastic behavior. The consideration of only elastic response of structural elements is sufficient for examining if structural deformations or stresses reach undesirable levels beyond the intent of the design.

Other interacting structural elements such as anchors, stabilization walls, lines, or soldier piles that are used to support the excavation, may be modeled using elastic elements or the rheologic model approach shown on Figure 4-2.

The model of the RB structure used for the non-linear FIA is sufficiently refined to:

a) adequately capture the interaction with the surrounding media,

b) properly develop all interfaces, and

c) properly transfer loads to and from the surrounding media.

4.3.3 Fluid-Soil Interaction

Groundwater elevations and hydraulic properties of the soils and rock are measured during the site soil and hydrogeological investigations as described in Section 3.0. The 3-D model developed for simulating stress-strain behavior of BWRX-300 may have hydraulic interface to simulate the effect of groundwater on the behavior of the structure during excavation, construction, loading and

operation. The model can simulate short-term as well as long-term dewatering or pumping as dictated by field conditions. The model simulates the changes in pore water pressures of the soil in response to unloading during the excavation stage and loading during construction and loading stages.

4.3.4 Analysis Staging Approach

Section 3.2 provides a description of the life stages of the BWRX-300, starting from the site investigation and ending with the plant operation. The BWRX-300 FIA are performed on numerical models that have the features to perform an integrated analysis of the stress, and deformation fields for each of the identified life stages:

4.3.4.1 Site Characterization

The FIA begins with the site itself, in its native condition, prior to any excavation or construction activities. During this stage, the initial stress conditions are aligned with the initial baseline displacement field. Initial stress conditions include, if applicable, the influence of groundwater aquifers and measured horizontal stresses.

4.3.4.2 Excavation

During the BWRX-300 RB shaft excavation, shown on Figure 4-4, soils and rock around and below the shaft may experience tensile stresses. The selected constitutive models allow for expansion response of soils resulting in heave or added pressures on excavation support structures. The changes in site conditions made prior or during the excavation are introduced in the FIA model following the sequence of the excavation plan. Non-linear interfaces are modeled between stabilization walls and soil.

As shown on Figure 4-5, the excavation simulation resembles the scheme planned for the specific site, by staging the removal of soil layers as excavation progresses and excavation support and site improvements are made. The stability of the excavation is verified in analytical space and later compared against field observations. The process allows for the design and monitoring of a safe excavation.

At the end excavation, the stress and displacement fields of the surrounding media, as well as the distribution of pore pressure, will have evolved. The “after excavation” condition is used as the initial condition for the analysis of the construction stage.

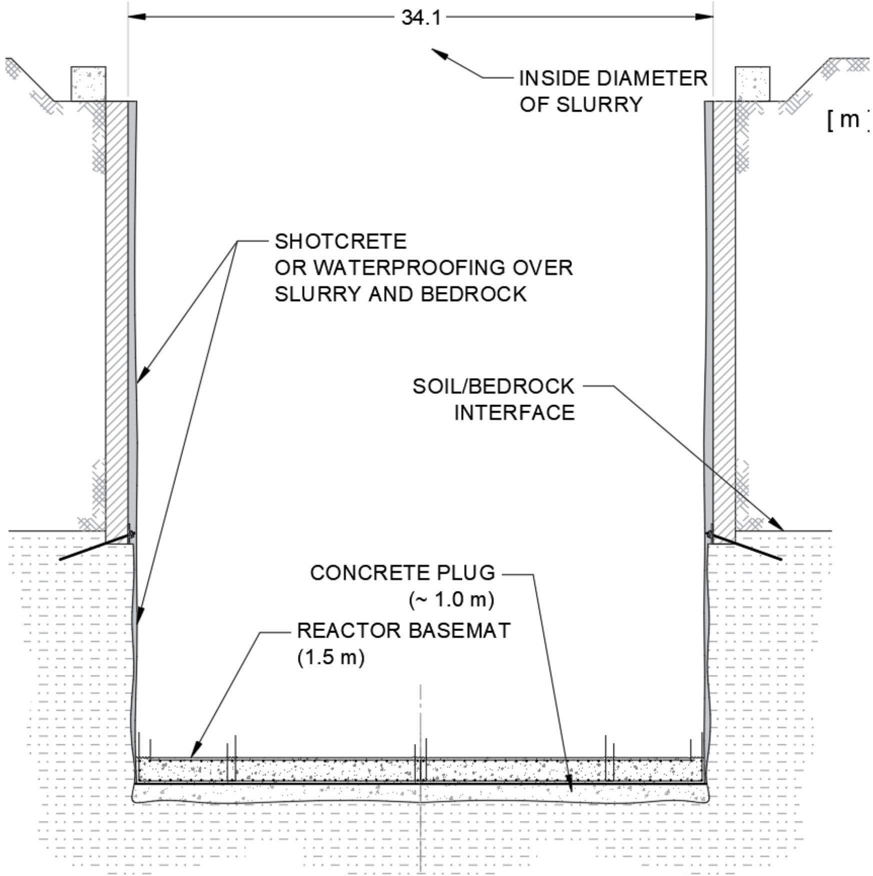


Figure 4-4: Excavation

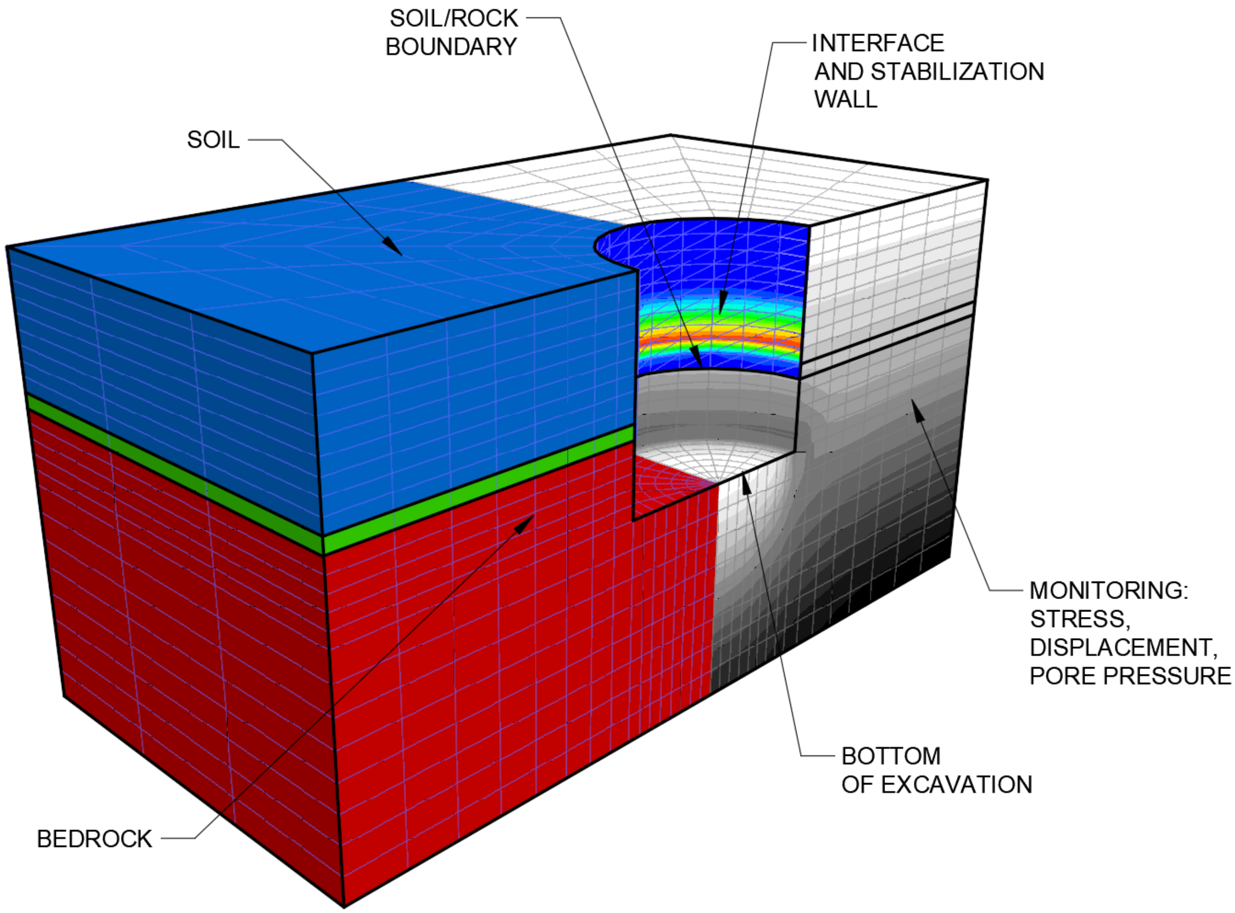


Figure 4-5: Excavation Modeling

4.3.4.3 Construction

Construction reloads the soil and rock and thus monitoring, and modeling comparisons continues as shown on Figure 4-6. The elastic and/or inelastic properties of soil and rock during reload are different from those used in the in-situ or excavation analysis. Therefore, the FIA uses deformation moduli that are consistent with the reloading stage.

The stability of structure, foundation and soil and rock subgrade is analyzed throughout the stages of construction by comparing field observations to estimated response obtained from modeling. Field observations are obtained from the field monitoring and construction inspection plan discussed in Sections 3.4 and 3.2, respectively.

Soil movement and potential joint displacement are continuously being analyzed by recomputing the equilibrium condition of the system. As equilibrium is reached, changes in soil stress and potential displacements in joints and interfaces still affect the response.

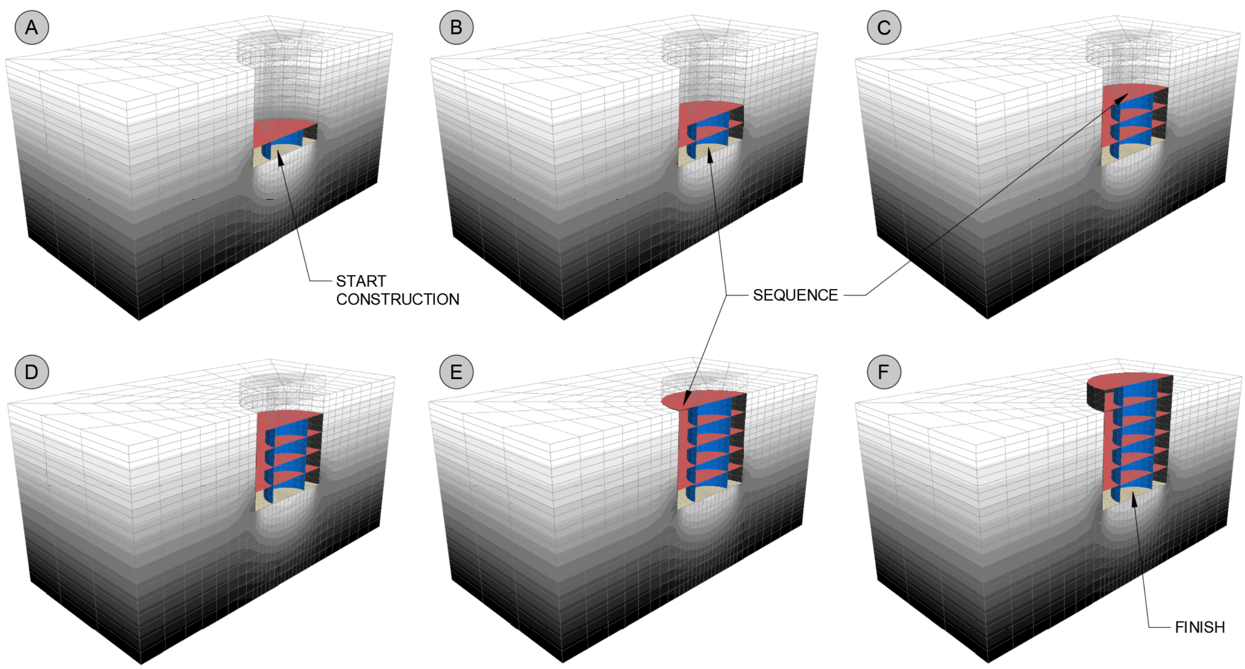


Figure 4-6: Modeling During Construction

4.3.4.4 Loading

Loading begins after the completion of the construction of civil structures and foundations and placement of the mechanical and electrical components, which introduce permanent dead loads. Other loads associated with the BWRX-300 operation are incorporated at this stage, such as weight of the fuel, water in the pools, cranes, and other permanent loads that are not previously introduced during construction. The CB, TB and RwB foundations around the RB are also included in the FIA model together with the surcharge loads.

FIA and monitoring of BWRX-300 RB response continues through the loading stage even though no significant movement is anticipated. The confirmation of the reduced response is critical for the safety of the plant because the additional loading may involve new components that possess stringent deformation limits.

4.3.4.5 Start-Up and Operation

New loads arise during the operational life of the BWRX-300, such as loads from ground motions, pressures and hazards from design flood and potential subsurface deformations that originate from subgrade instabilities. Sensitivity studies may be performed to analyze potential formation of instabilities in the subsurface, as shown on Figure 4-7, or to investigate the effect of flooding. These analyses allow the determination of effects on the structures that arise from new loadings during operation. Changes in the exerted forces due to non-linear response of the surrounding media can be assessed.

In the FIA space, most physical properties for any point within the model can be closely monitored as shown on Figure 4-8. Responses of interest of discrete points within the soil are mainly displacements, stresses, and pore pressures. Stress distribution and plastic zones can be monitored

throughout the zones within the model. For actual conditions in the field, monitoring is limited by the number of sampling points with sensors/monitoring equipment and the limitations of the instrumentation equipment. Sections 3.3 and 3.4 present a description of the instrumentation plan.

The elements modelling improvements, such as consolidation grouting, rock reinforcement, and soil support, which are made during the excavation stage are kept in the FIA model to account for their effects when monitoring subgrade conditions during start-up and operation. However, these improvements are removed from the FIA model for the purposes of earth pressure design load validations described in Section 5.1.3.

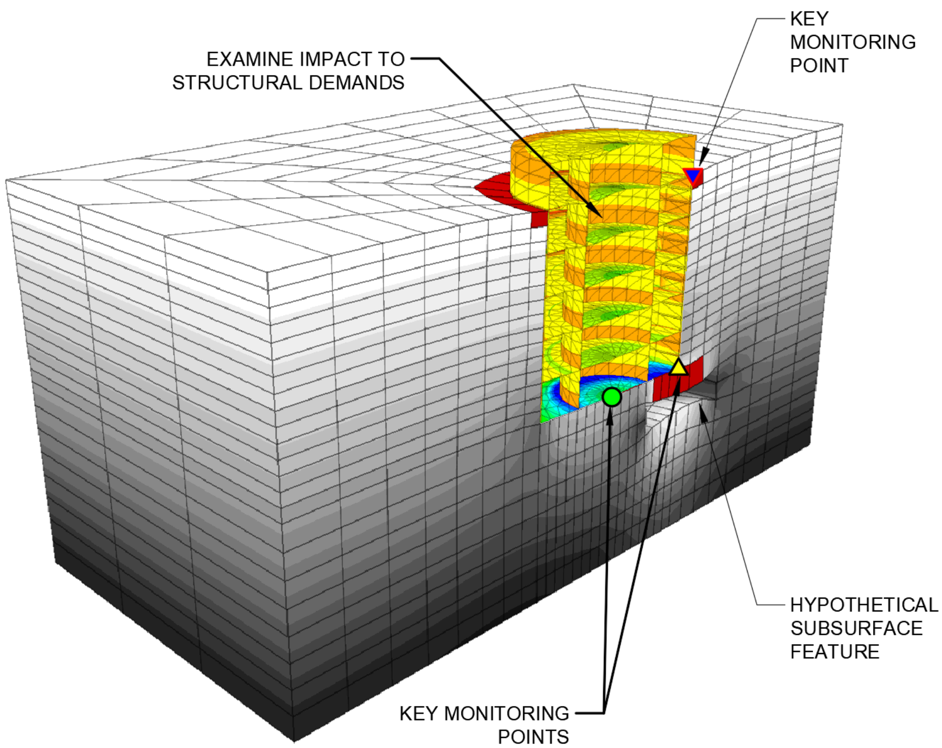


Figure 4-7: Modeling During Operation

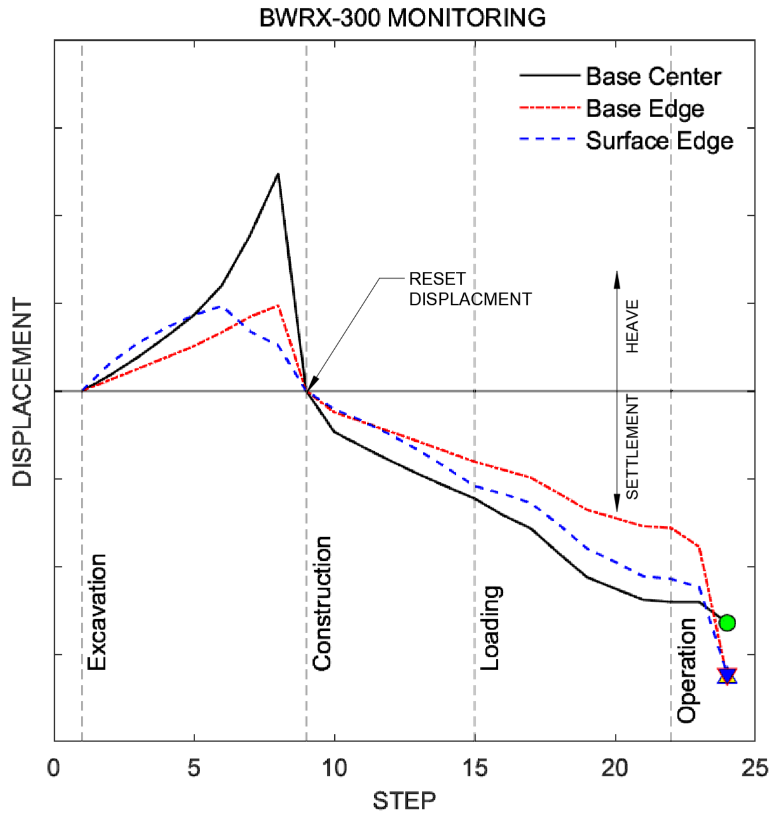


Figure 4-8: Hypothetical Results of Modeling During Operation

4.4 Summary of Foundation Interface Analysis

The following aspects of the innovative approach implemented for the BWRX-300 FIA presented in this section of the report, may be referenced during future licensing activities:

(1) General modeling and analysis requirements provided in Section 4.1 that are beyond the current regulatory guidance of SRP 2.5.4 and industry practice for NPP stability evaluations.

(2) Guidelines for modeling the non-linear constitutive response of soil and rock materials, presented in Sections 4.2.1 and 4.2.2, respectively, including the approach for calibrating the FIA model based on data obtained from field instrumentation described in Section 3.4 that are beyond the guidance of SRP 2.5.4.

(3) Guidelines for modeling interfaces presented in Section 4.3.1, including contacts between structures and the subgrade, as well as interfaces between bedding units, faults and joint planes in the geological formation that are beyond the guidance of SRP 2.5.4.

(4) FIA structural modeling requirements provided in Section 4.3.2, including recommendations for modeling SMR structures and soil stabilizations elements, such as rock anchors, soldier piles, and stabilization walls and liners that are beyond the guidance of SRP 2.5.4.

(5) FIA modeling approach for fluid-soil interaction presented in Section 4.3.3, and FIA model calibration using measurements of groundwater elevations and hydrogeological investigations described in Section 3.0 that are beyond the guidance of SRPs 2.5.4 and 3.8.5.

(6) FIA approaches for the different BWRX-300 life stages presented in Section 4.3.4, including guidelines for using the measurements from field instrumentation described in Section 3.4 for FIA model calibration and benchmarking FIA results that are beyond the guidance of SRPs 2.5.4 and 3.8.5.

5.0 DESIGN ANALYSES

Innovative static and seismic SSI analysis approaches for designing the deeply embedded RB structure are presented in Sections 5.1 and 5.3, respectively. These approaches address the BWRX-300 design and construction specifics related to soil column interaction with the deeply embedded SMR structures identified in NUREG/CR-7193, Section 1.5.11 (Reference 8.1). Section 5.2 presents the requirements, methodologies, and recommendations for developing site-specific geotechnical and seismic design parameters based on the results of site investigations and laboratory testing programs described in Section 3.1.

Requirements and recommendations are provided in Sections 5.2.2 and 5.3.4, respectively. These requirements and recommendations ensure the seismic SSI analyses use input motion that is adequate throughout the depth of the RB embedment. These requirements are beyond the current regulatory guidance and address specifics related to the seismic analysis of deeply embedded structures identified in NUREG/CR-7193, Section 1.5.8.

A comprehensive recommended approach is provided in Section 5.3.3 for evaluating the effects of non-vertically propagating seismic waves on the design ground motion and seismic response of the deeply embedded RB structure. This approach is beyond the current regularly guidance and addresses an important issue for the deeply embedded SMRs design, identified in NUREG/CR-7193, Section 1.5.9.

Section 5.3.6 recommends approaches for developing in-structure seismic response demands for equipment design and qualification, considering ESI. These approaches are beyond the current regulatory guidelines and address issues related to ESI effects on design of equipment with more complex dynamic behavior that are identified in Section 1.5.2 of NUREG/CR-7193.

Additional requirements are introduced in Section 5.2.3 for generating acceleration time histories that are used as input to the seismic SSI analyses that are beyond the current regulatory guidance. These requirements ensure mitigation of uncertainty in the computed responses due to the phasing of the time history frequency components and the accuracy of the calculated high-frequency in-structural responses.

5.1 One-Step Design Analysis Approach

As described in Section 1.3, almost all of BWRX-300 important to safety SSCs that are required to maintain their structural integrity and safety functions during and after a SSE event are hosted in the RB that is classified per regulatory guidance of SRP 3.2.1, "Seismic Classification," Revision 3, as SC-I structure. Most of the SC-I SSCs including the BWRX-300 RPV and the CPV are located below grade of the RB.

Because a significant part of the RB structure is located below grade, the interaction of the structure with the surrounding soil is a very important factor for the integrity of the RB structure and its response under static and dynamic loads. The below grade portion of the RB structure is subjected to static and dynamic loads from:

- self-inertia loads including loads from equipment, and pool water;
- the mass and impedance of the surrounding in-situ subgrade materials,

- groundwater hydrostatic pressure; and
- overburden loads and the interaction with the surrounding RwB, CB and TB foundations and structures.

Furthermore, the interaction with the surrounding subgrade determines the boundary conditions at the RB below-grade shaft exterior wall and basemat interfaces thus affecting the structural response and stress distribution from other static and dynamic loads such as operating and accidental thermal and pressure loads.

In order to adequately account for the SSI effects, the one-step approach, as defined in Section 3.1.2 of ASCE/SEI 4-16 (Reference 8.7), is implemented for the design of the BWRX-300 RB structure using a linear elastic SASSI (a system for analyses of soil-structure interaction) analysis approach described in Section 5.3. Static and dynamic structural stress demands are obtained directly from the results of SSI analyses of combined models that include FE representations of the RB structure and the surrounding soil. The surrounding subgrade is represented by layered half-space continuum with equivalent linear elastic stiffness properties and complex damping.

Stress demands on the RB structural members due to static earth pressure, structural self-weight, equipment weight and live loads are calculated by applying 1-g gravity loads on the combined model of the RB structure and the subgrade continuum. The structural demands due to overburden pressures from the nearby foundations are also calculated by the 1-g static analysis. Additional static analyses are performed to calculate the structural demands due to hydrostatic wall pressures from the pool water, normal operating and accidental pressure loads. Separate analyses provide the structural demands due to normal operating and accidental pressure and thermal loads. Structural demands due to seismic inertia loads and dynamic soil pressure loads are obtained from seismic SSI analyses that are described in Section 5.3.

The methodology used for development of RB FE model is based on the methodology described in Section 5.1.1 and the SSI modeling assumptions presented in Section 5.1.2. Equivalent linear properties are used as input for the static and seismic SSI analyses developed as described in Sections 5.2.1 and 5.2.4, respectively. Section 5.1.3 presents the unique BWRX-300 approach used to demonstrate that the linear-elastic SSI analyses provide soil and rock pressure load demands with sufficient design load margins to address the modeling uncertainties.

5.1.1 FE Model of RB Structure

The structural FE model consisting of beam, shell, solid, and spring elements adequately represents the RB structural configuration for all main structural members. The FE model includes gross discontinuities such as large openings and member eccentricity. Thick shell elements are used to model the reinforced concrete shear walls, slabs and basemat. 3-D beam elements are used to model the reinforced concrete or steel columns, beams, and trusses. The shell and beam elements are established at the centerline of the wall, slab, beam, column, and truss elements. Rigid beam and shell elements or rigid links are used to model member eccentricities and offsets.

Linear elastic contact springs connect the RB structural and subgrade FE models. Stiffness properties are assigned to the contact springs to adequately represent the interaction mechanism between the structure, the water proofing material and the soil as described in Section 5.1.2.

Results obtained from these contact spring elements serve for calculation of soil pressures on the below grade RB shaft exterior wall. The results obtained from the contact spring elements serve to:

- validate the earth pressure loads considered by the design as described in Section 5.1.3, and
- determine whether separation between RB shaft wall and soils occurs in the static and dynamic loadings as discussed in Section 5.3.9.

The mesh of the FE models is sufficiently refined to produce stress demand calculations that are not significantly affected by a further refinement of the FE size or the shape. Finer meshes are used around penetrations and openings that are larger than half of the wall or slab thickness. Meshes of major walls and slabs consists of at least four shell elements along the short direction and at least six shell elements along the long direction.

The FE models used for seismic SSI analyses have a sufficiently refined mesh to be capable of transmitting the entire frequency range of interest for the seismic design of the RB SSCs. In accordance with the requirements of ASCE\SEI 4-16 (Reference 8.7), Section 5.3.4, the FE mesh shall be smaller than or equal to one-fifth of the smallest wavelength transmitted through the soil model, i.e. the maximum mesh size:

$$d_{max} \leq \frac{V_S}{5\, f_{cutoff}} \quad (5\text{-}1)$$

where: V_S is the shear wave velocity of the transmitting soil material; and

f_{cutoff} is the cutoff frequency of analysis determined as described in Section 5.3.2

Larger element sizes may be used when justified as described in Section 5.3.4 of ASCE\SEI 4-16. Stiffness properties are assigned to structural members in the RB FE model in terms of Young's modulus and Poisson ratio that are determined in accordance with the governing design codes:

- American Concrete Institute ACI-349-13 (Reference 8.24) for the reinforced concrete members; and
- AISC N690-18 (Reference 8.25) for the steel and steel-plate composite (SC) members.

5.1.2 Soil-Structure Interaction Modeling Assumptions

Several simplified assumptions are introduced in the SSI design analyses of RB FE model to enable an efficient calculation of stress demands on the RB structure due to pressure loads from soil and rock surrounding and supporting the RB shaft. The following are the main assumptions for subgrade modeling used for the design SSI analyses performed following the SASSI methodology:

1) The properties of the subgrade materials are assumed to be isotropic and linear elastic;
2) The non-linearities at soil-structure interfaces are neglected;
3) The rock mass is assumed continuous and the presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones is neglected;

4) The static lateral pressures on the RB shaft due to the weight of self-supporting rock (i.e., excavated rock that does not require lateral support) can be neglected.

As described in Section 5.2.1, an approach is used for the development of linearized properties of soil and rock materials for the 1-g static SSI analysis to provide upper bound estimates of the demands on the RB structural members. Upper bound structural deformations and stress demands and lateral soil pressures on the RB below-grade exterior walls are estimated by using upper bound values for the soil unit weight and soil and rock Poisson's ratio paired with lower bound values of soil and rock elastic moduli.

The following stiffness properties are assigned to the contact springs at the SSI interfaces in the RB FE model for 1-g design analysis to provide upper bound lateral soil pressures on the RB below-grade exterior walls:

- The contact springs in the direction normal to the RB exterior walls are assigned properties representing upper bound stiffness conditions at the SSI interfaces; and
- The friction at the RB exterior walls is neglected by assigning very low stiffness properties to the contact springs in vertical and tangential direction.

The soil and rock strata in the SSI models used for calculating demands for design of RB structure are modeled based on the principles of continuum mechanics using isotropic linear elastic properties. Possible fracture zones, joints, bedding planes, discontinuities and cavities in the rock are not explicitly included in the design SSI analyses models. The stiffness properties assigned to the rock materials are developed, as described in Section 5.2.1.2, using empirical engineering and geomechanical rock mass classifications that quantitatively characterize the geologic and engineering parameters of rock masses.

The approaches described in Section 5.2.1.2 to calculate the equivalent linear properties of rock are applicable to structures that are relatively large compared to the block size of the rock mass and assumes the closely spaced discontinuities have similar characteristics where isotropic behavior of the rock mass is valid. When the discontinuity spacing is large compared to the dimensions of the excavation, the potential for unstable blocks or wedges and swelling or squeezing rock units need to be evaluated. The size of potentially unstable rock blocks and wedges should be estimated using an appropriate method (e.g., Reference 8.69). The evaluation of the potential loads from rock blocks and wedges may be completed using:

- the nonlinear FIA that includes rock/rock discontinuities represented by interface models described in Section 4.3.1.2; or
- static or pseudostatic force equilibrium analysis.

A simple example of a model for force equilibrium analysis of rock stability is provided in Section 5.1.4.3.

Strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness may be self-supporting even if some reinforcement is required to ensure a safe excavation. Typically, rock masses will yield slightly during construction – even with well-placed reinforcement – and arching will reduce the lateral loads except in highly fractured, weak, swelling, or squeezing rocks.

Because it is much more economical to reinforce the rock mass than to support it, rock reinforcement is used to create a self-supporting rock mass when the natural rock mass is not self-supporting. Reinforcement like tensioned and untensioned anchors may be installed inside the rock mass to help the rock mass support itself by eliminating progressive failure along planes of low strength as described in USACE 1110-1-2907 (Reference 8.26). Frequently, the reinforcement addresses specific rock wedges (keying) or is designed to form a beam or arch within the rock to create a stable, self-supporting excavation. Surface treatments such as shotcrete, strapping, and mesh may also be used for stabilization, protection of exposed rock, and control of loosened rock.

The design of the BWRX-300 considers this rock reinforcement as initial ground support that is separate from the permanent ground support system because the rock reinforcements and any surface protection may be inaccessible after construction. Therefore, the design addresses the rock loads remaining after the initial ground support degrades by including the potential weight of the solid rock in the design 1-g SSI analysis based on the results of non-linear FIA as described in Section 5.1.3.

Additional design analysis may be performed where earth pressure loads are applied to the below grade exterior walls of the refined RB structural model to account for:

- the effects on the RB design of anisotropic or heterogenous rock responses that cannot be directly modeled by the isotropic elastic models used for the one-step design SSI analysis; or
- potential pressures from unstable blocks of rock mass.

The magnitude and distribution of these additional earth pressure loads are determined from the results of the nonlinear FIA or force equilibrium analyses of the unstable rock mass. The structural design demands obtained from this additional earth pressure analysis are combined with the results of the one-step SSI analysis to ensure the RB structural design adequately addresses the effects of anisotropic and heterogenous rock behavior and accounts for potential unstable rock mass loads.

The SSI analysis of RB FE model are performed for a set of subgrade profiles to account for the variability and uncertainties in the subgrade material properties in accordance with the regulatory guidance of SRP 3.7.2 Subsection II.4 and ASCE/SEI 4-16 (Reference 8.7), Section 5.1.4. To address the effects of primary non-linearity, soil dynamic properties are used that are compatible to the free-field strains generated by a typical design level earthquake. These strain-compatible properties are developed as described in Section 5.2.4.

The effects of secondary non-linearity induced in the soil and rock by the structural vibration are neglected because in general, the structural vibration induces plastic deformations of the soil and dissipation of energy in the SSI system that reduces the structural response as shown in Reference 8.27 and Reference 8.28. On the other hand, the secondary non-linearity of subgrade materials may amplify the magnitude of the dynamic lateral pressures. The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones within the rock mass may also affect the stability of individual blocks or the rock mass during an earthquake that can potentially amplify the seismic rock pressure loads. Section 5.3.11 describes the approach used to evaluate the effects of subgrade materials non-linearity on the seismic response and design

BWRX-300 RB when it is constructed at sites characterized with a high non-linear behavior of the subgrade materials and high seismicity.

The design basis seismic analyses of BWRX-300 RB are performed on models that assume fully bounded conditions at the interfaces between the RB structure and the subgrade. Depending on the subgrade conditions and the intensity of the design ground motion, separations may occur at the SSI interfaces during an earthquake event. Section 5.3.9 describes a conservative approach for addressing these effects of soil separation on the RB seismic response and design.

5.1.3 Design Earth Pressure Load Validation

Section 4.0 describes the FIA performed on numerical models representative of the non-linear constitutive behavior of soil and rock materials surrounding the RB shaft and employ non-linear interface modeling features capable of capturing the effects of non-linearities at the soil-structure contact surfaces. The model also includes the main structural elements of the RB that adequately represents the stiffness properties of the structure interacting with soil and accurate calculations of the contact pressures at soil-structure interfaces.

Results for maximum soil and rock pressure loads on the RB exterior walls obtained from the FIA and the linear elastic 1-g design analysis are compared to:

- assess the effect of non-linear and anisotropic behavior of subgrade materials on the soil and rock pressure demands;
- demonstrate that the SSI modeling assumptions listed in Section 5.1.2 yield conservative design demands; and
- assess the conservativism of the soil and rock pressure demands obtained from the 1-g design analysis for the design of RB structure.

As described in Section 4.3.4, the FIA considers staged excavation, construction and loading sequences to adequately model the change in in-situ stress due to construction activities and establish the initial conditions for calculation of soil pressures at the stage when the plant is in operation. However, detailed stages of excavation and construction as presented in Section 4.4 are not required for the soil and rock pressure loads validation. Stages like excavation and construction may be completed in a single step instead of multiple steps because the monitoring details are not required.

The validation of soil and rock pressure loads may consider the subgrade improvements like consolidation grouting, rock reinforcement, and soil support made during the construction. However, these improvements are considered only as initial ground support that is separate from the permanent ground support system because these types of reinforcements and any surface protection will be inaccessible for monitoring and repair after the construction. Therefore, unimproved soil and rock conditions are considered due to the uncertainty in:

- the long-term durability of grout, as noted in Paragraph 2-5 of USACE EM 1110-2-3506 (Reference 8.29);
- potential degradation of rock reinforcement, as noted in USACE EM 1110-1-2907 (Reference 8.30); and

- degradation of other soil support system.

This additional rock load on the RB shaft wall may be uniform with contact grouting to avoid stress concentration or point load associated with the block or wedge that is reinforced to stabilize the rock excavation. The evaluation of these rock pressure loads assumes that the excavation has reached stability with initial rock support and that the liner will accept 100 percent of the initial rock support as it relaxes over the lifetime of the structure. These loads should be conservative because rock loads in stressed rock masses are typically not following (e.g., they are not independent of displacement and typically reduce with displacement due to arching). The notable exception would be due to the presence of hydrostatic loads and swelling or squeezing rock displacements that may continue to apply a large load with continued displacement.

The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. This potential load transfer is dependent on the geometry of the discontinuities, surface structure and embedded structure. When the additional load from the surface structure may be transferred to a potentially unstable rock block or wedge, this additional load should be included in the determination of reinforcement and the potential rock load on the exterior of the shaft or the rock block or wedge may be over-excavated and backfilled to reduce the load. Consideration of the geometry of the load transfer may allow the surface structures to be re-arranged to reduce or eliminate this load transfer to a potentially unstable rock block or wedge.

If cavities are present at the deployment site, sensitivity analysis are also performed by varying locations and sizes of cavities to address the effects of potential cavities on the rock pressure demands on the RB structure during operation.

The pressure load validation FIA uses the constitutive models described in Section 4.2 to represent the non-linear response of soil and rock subgrade materials, and the models described in Section 4.3.1 to represent the response at interfaces including the interfaces of RB structure with the surrounding subgrade. Because the intent of the FIA is to calculate best estimates of the soil and rock pressure loads, constitutive and interface models are developed using best estimate soil and rock properties obtained from the results of site investigation and laboratory testing programs described in Section 3.1. The stiffness of the RB structure in the FIA models is calculated per the governing design codes. Conservative design values obtained from the literature can also be used for certain input parameters.

A best estimate soil and rock pressure profile on the RB shaft is developed as an envelope of all maximum lateral pressure values calculated by the non-linear FIA of all analyzed post-construction stages and scenarios. This lateral pressure profile is compared to the lateral pressure profile developed from the results of the linear elastic 1-g design analysis to confirm the equivalent linear elastic model provides adequately conservative loads for the structural design. Soil and rock design pressure margins are calculated based upon the minimum values and the distribution of the ratio between the design soil and rock pressures obtained from the 1-g linear elastic analysis and the best estimate pressures obtained from the non-linear FIA. If the values of the calculated soil and rock design load margins are below the values deemed adequate to address the uncertainties and variations of subgrade properties, the rock mass weight or the equivalent linear soil and rock stiffness properties used for the 1-g design analysis are adjusted. Adequate values of the soil and rock design load margins are established based on the uncertainties and

variability of soil and rock properties used as input for the non-linear FIA and the significance of the non-linear and anisotropic response of subgrade materials on the soil and rock pressure demands.

If the results of non-linear static FIA indicate that the non-linear and anisotropic effects have a significant effect on the rock soil pressures and the site is characterized by a high seismicity, sensitivity SSI analyses are performed on non-linear models, as described in Section 5.3.11, to assess the effects of non-linear soil and rock response on the dynamic lateral pressure demands.

5.1.4 Probabilistic Earth Pressure Analyses

Probabilistic analyses may be performed to demonstrate that the magnitude of earth pressures used for the design are adequate to address uncertainties in the pressure load calculations. The external wall of the RB that is contact with soil is subdivided into discrete regions. The general approach consists on computing the probability density function of the subgrade pressure at each discrete region to calculate the probability distributions of soil and rock pressure loads on the RB below-grade exterior walls.

The probabilistic earth pressure load analysis addresses two types of uncertainties in the calculations of earth pressure loads:

- Parameter uncertainties related to natural randomness and uncertainties in measurements of mechanical properties of in-situ subgrade materials; and
- Model uncertainties related to the models used for earth pressure calculations.

Parameter uncertainty includes random variability of measured parameters including spatial variability and systematic measurement errors as well as uncertainties related to the methods used for the development of site subgrade parameters from empirical relationships. The random variability is manifested as the scatter of the data around a mean trend and is composed of the spatial variation of the subgrade properties and random measurement errors. Because the random measurement errors are often not distinguishable from spatial variation of the subgrade properties, they are usually considered jointly. Systematic error is divided into:

- Statistical error in the mean that can be reduced with increasing the sample size and number of measurements and tests being performed
- Bias in sampling and measurement procedures that is corrected by means of correction techniques/algorithms
- Bias introduced by the method used for development of subgrade parameters that is addressed by considering different approaches and empirical equations to calculate discrete probability distributions that are then combined as described in Subsection 5.1.4.4.

The model uncertainty that represents the uncertainty related to the model's ability to accurately predict the soil and rock pressures is manifested as a bias error in the earth pressure calculations. In general, the model uncreate is reduced by using more sophisticated models and an increasing number of model parameters. On the other hand, the increasing number of parameters used in the sophisticated models increases the parameter uncertainty and may reduce the overall confidence in the calculated soil pressure results. The model uncertainty is approached by means of

considering different models that utilize fewer input parameters resulting in discrete probability distributions that are combined as described in Subsection 5.1.4.4.

5.1.4.1 First Order Second Moment Method

The First Order Second Moment (FOSM) method may be used for simple calculations of the probability density function of the ground pressure. Following the approach described in (Reference 8.31), earth pressures (P) at each discretized region are represented by the following function:

$$P = g(x_1, x_2 \dots x_n) + e \quad (5\text{-}2)$$

where: g represents a geotechnical multivariable function of the earth pressure at a discretized element

$x_1, x_2 \dots x_n$ are the site parameters whose variation has an important effect on the earth pressures

e represents the biased modelling and measurement systematic errors.

The probability calculations may consider other parameters than the random parameters $x_1, x_2 \dots x_n$. These parameters where variations have relatively insignificant effects on the earth pressures, may be considered deterministically using values that ensure a reasonably conservative bias in the results of the probabilistic analyses.

The mean value of the earth pressure ($\bar{P}$) is expressed as function of the mean values of the site parameters ($\bar{x}_1, \bar{x}_2 \dots \bar{x}_n$):

$$\bar{P} = g(\bar{x}_1, \bar{x}_2 \dots \bar{x}_n) \quad (5\text{-}3)$$

For a sample of $1, 2 \dots m$ measurements, the mean values of each parameter $\bar{x}_i$ in Equation (5-3) are calculated as follows:

$$\bar{x}_i = \frac{1}{m} \sum_{k=1}^{m} (x_{ik}) \quad (5\text{-}4)$$

where: x_{ik} is the kth measured data point of the parameter x_i.

The mean values of the earth pressures (P) are calculated either by using the simplified models described in Subsection 5.1.4.3 or from the results of non-linear FIA that use inputs based on best estimates or mean values of the site parameters as described in Section 5.1.3.

The variance of the earth pressures $V[P]$ can be expressed by using the Taylor expansion described in section 12.4.3.2.2 from NUREG/CR-2300 (Reference 8.32):

$$V[P] \approx \sum_{i=1}^{n} \sum_{j=1}^{n} \frac{dg}{dx_i} \frac{dg}{dx_j} C[x_i, x_j] + V[e] \quad (5\text{-}5)$$

where: dg/dx_i is the derivative of $g(x_1, x_2 \dots x_n)$ with respect to parameter x_i;

$C[x_i, x_j]$ is the covariances among parameters i and j,

$C[x_i, x_i] = \mathrm{V}[x_i]$ is the variance of parameter x_i;

$V[e]$ is the variance related to the bias errors.

The $C[x_{i,} x_j]$ terms in Equation (5-5) representing the variances and covariances of the input parameters defining the mechanical properties of the subgrade materials, such as the cohesion, internal friction angle, are determined based on the statistical analysis of the results of site investigations and laboratory tests described in Section 3.1. The variance of parameter x_i can be calculated as follows:

$$C[x_i, x_i] = V[x_i] = \frac{1}{m-1}\sum_{k=1}^{m}(x_{ik} - \bar{x}_i) \qquad (5\text{-}6)$$

The covariance $\mathrm{C}[x_i, x_j]$ of the two parameters x_i and x_j is calculated as follows:

$$C\left[x_i, x_j\right] = \frac{1}{m-1}\sum_{k=1}^{m}(x_{ik} - \bar{x}_i)\left(x_{jk} - \bar{x}_j\right) \qquad (5\text{-}7)$$

where: x_{ik} and x_{jk} are the k$^{\text{th}}$ measured data points of the parameters x_i and x_j, respectively;

$\bar{x}_i$ and $\bar{x}_j$ are the mean values for the parameters x_i and x_j, respectively.

If the information obtained from the site investigations and laboratory testing is not sufficient, as described in Reference 8.33, some degree of belief probability based on engineering judgment may be used to establish variances and covariances for some input parameters.

Earth pressure derivatives dg/dx_i for each parameter x_i in Equation (5-5) are calculated for each discretized region. According to the model used to describe parameter x_i, these derivatives are calculated either analytically or numerically. If the relation between the earth pressure and parameter x_i is defined by an analytical function of that parameter, dg/dx_i is calculated as a derivative of the function with respect to the variable x_i. For example, if the Jacky's coefficient at rest in equation (5-15) is used to corelate the earth pressure P to the soil friction angle (φ)

$$P = \gamma\, z\,(1 - \sin\varphi) \qquad (5\text{-}8)$$

where: γ is the unit weight of the soil material

z is the depth of the zone where the pressure is calculated from the ground surface.

the value of dg/dx_i is obtained from the derivative of Equation (5-8) with respect of φ as follows:

$$\frac{dg}{d\varphi} = \bar{\gamma}\, z\,(1 - \cos\bar{\varphi}) \qquad (5\text{-}9)$$

where: $\bar{\gamma}$ and $\bar{\varphi}$ are the mean values of the soil density and friction angle, respectively.

The following expression is used to numerically calculate the derivative of $g(x_1, x_2 \dots x_i \dots x_n)$ with respect to the parameter x_i:

$$\frac{dg}{dx_i}(x_1, x_2 \dots x_i \dots x_n) = \frac{\boldsymbol{g}(\bar{x}_1, \bar{x}_2 \dots \bar{x}_i \dots \bar{x}_n) - \boldsymbol{g}(\bar{x}_1, \bar{x}_2 \dots \bar{x}_i + \Delta x_i \dots \bar{x}_n)}{\Delta x_i} \quad \text{(5-10)}$$

where: Δx_i is adequately selected small change in the variable x_i value.

The FOSM method may not be applicable when the geotechnical function $g(x_i)$ relating the relationship between the parameter x_i and the earth pressure P is highly non-linear. An example is the case when the mean value of pressure on the RB shaft arising from instabilities of the rock mass due to discontinuities is zero because the rock mass is stable in the mean probability case. For this case, the Monte Carlo Method described in Section 5.1.4.2 may be used or a conservative bias in the probabilistic earth pressure calculations may be introduced by defining the mean value of the rock pressure to be a positive value when combining the parameters in Equation (5-5).

5.1.4.2 Monte Carlo Method

The Monte Carlo method, described in Section 12.4.3.1.3 of NUREG/CR-2300 (Reference 8.32), may also be applied to assess the probability distribution of the earth pressures without using FOSM. A set of at least 60 randomized realizations are generated for each parameter, whose variation has an important effect on the earth pressures, according to their probability distribution. The generated random parameter realizations are then used to calculate a sample of at least 60 random earth pressures, whose distribution is adopted as their probability distribution used to calculate the probability of soil and rock pressure loads to exceed the design pressure loads.

The Monte Carlo analysis is relatively simple and easy to implement when the relationship between parameters and wall pressure can be described by simple analytical equations or analytical models. On the other hand, Monte Carlo probabilistic non-linear FIA are complex and computationally demanding analyses.

5.1.4.3 Probabilistic Analysis Earth Pressure Models

Every parameter$(x_1, x_2 \dots x_n)$ in Equation (5-5) whose variation has an effect on earth pressures is related to earth pressures through a model for each discretized region. This model may be:

- an analytical model based on plasticity theory, limit equilibrium method solutions or empirical equations;
- a force equilibrium model; and
- a FE model or a finite difference model.

Table 5-1 summarizes the different site parameters and types of models that are commonly used in the probabilistic analyses of earth pressures, in particular for the FSOM calculations to obtain the values of parameter derivatives dg/dx_i.

Table 5-1: Models for Probabilistic Earth Pressure Analyses

Subgrade Type	Site Parameter (x_i)	Model
soil	unit weight	Analytical equations
	cohesion	
	friction angle	
rock	rock mass properties	Force equilibrium, FE or a finite difference model
	unit weight	
	cohesion	
	friction angle	
	weak zone orientation	
	weak zone area	

Simple models that do not require explicit calculations of the state of strain and stress in the ground materials, are used for the probabilistic analyses of earth pressures on the RB shaft in contact with subgrade materials which mechanical properties are assumed to be continuous. For example, the following three models can be used to calculate lateral earth pressure coefficients representing three possible states:

a. at-rest condition representing essentially no movement of the structure relative to the surrounding subgrade;
b. active condition when the structure moves away from the surrounding subgrade; and
c. passive condition when the structure moves towards the surrounding subgrade.

These simple models provide probabilistic earth pressure distributions from the probabilistic distributions of the basic subgrade material strength parameters, the internal friction angle (φ), the cohesion (c) and the friction angle (φ_w) between the subgrade and RB cavity wall.

Force equilibrium models are used for probabilistic analysis of rock masses with discontinuities that may control the stability of individual blocks or the rock mass when the orientation is disadvantageous. Depending on the geometry of the discontinuities relative to the free face of the excavation, one or more blocks may slide along the discontinuities.

As shown on Figure 5-1, the sliding of the rock block driven by the surcharge load and its own weight is resisted by:

- the resistance force along the rock discontinuity due to cohesion (c_d) and the friction represented by the friction angle (φ_d); and
- the resultant of pressure loads at the rock-structure interface.

If the rock mass is located below the ground water table, the rock mass stability is also affected by the effects of groundwater buoyancy.

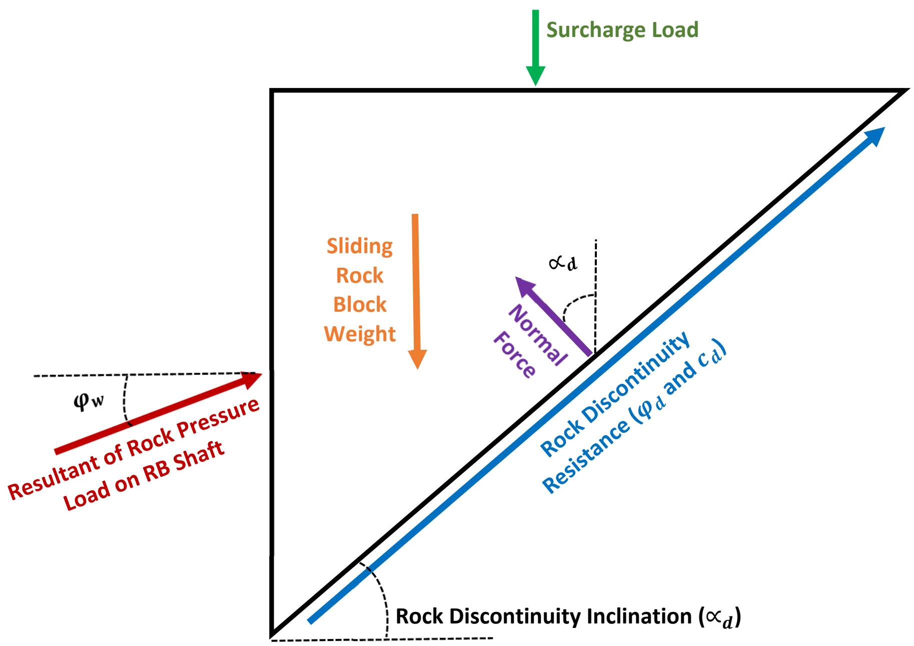


Figure 5-1: Force Equilibrium Model for Rock Wedge Analysis

The probability distribution of the rock pressure load demand on the RB shaft is obtained by considering the equilibrium between the driving and the resisting forces and the probability distributions of the input parameters, such as the friction (φ_d) and cohesion (c_d) of the rock discontinuity, the discontinuity inclination angle ($\propto_d$), rock-wall friction angle (φ_w).

Figure 5-1 presents a simple rock wedge model to illustrate the concept. More complex models regarding both the geometry of sliding block and the resistance parameters may be used that also consider the spatial distributions of the input parameters. In general, the orientation of rock discontinuities is three-dimensional. The uncertainties related to the location, extent and orientation of the rock discontinuities are usually addressed as parameter uncertainties using fore equilibrium models representative of different rock discontinuity configurations. The parameters and models for calculation of rock pressures and their probabilities are determined based on engineering judgment using the results of the field investigations and the FIA, described in Section 3.1.1 and Section 4.0, respectively.

5.1.4.4 Combining Discrete Probability Distributions

The use of different models for the probabilistic earth pressure calculations leads to discrete probability distributions that need to be combined in a continuous earth pressure probability distribution. To illustrate the process of combining these discrete probability distributions to a continuous distribution, a simple example is used for calculation of the probability distribution of lateral pressures on the RB shaft from a relatively uniform soil that is based on the use of two

simple models, the Rankine active soil pressure (P_A) equation and the Jacky's empirical at-rest soil pressure (P_0) equation.

Based on knowledge and judgment reflected by the degree of belief probabilities, an estimate of the probabilities for the soil to be active and at-rest condition can be made, for example, 30% for P_A and 70% for P_0. These belief probabilities can be estimated based on the knowledge gain from the non-linear FIA results using a lottery or a probability wheel.

Monte Carlo method may be used to combine the discrete belief probabilities with the parameter uncertainties. Mean values and variance values for of active pressure P_A and at rest pressure P_0 are calculated using the FOSM Equations (5-3) and (5-5), respectively. The input values of mean, variance and covariance for the soil friction angle and cohesion coefficient are obtained from Equations (5-4), (5-6) and (5-7), respectively, using field and laboratory tests measurements. Monte Carlo simulation is used to calculate two sets of at least 60 random realizations for the active pressure P_A and at rest pressure P_0 based on their probability distributions. Another set of 60 random numbers are generated with value ranging from 0 to 1 ($R_{BP} = 0 \text{ to } 1$) to simulate the random discrete probabilities for the active P_A condition or the at-rest P_0 condition where applicable. If the value of $R_{BP} \leq 0.3$, the calculated random P_A is adopted as the soil lateral pressure realization, otherwise, the calculated P_0 value is stored in the sample of calculated soil lateral pressure realizations that combine the discrete belief and parameter probabilities. The calculated sample of 60 or more soil pressure random realizations is then used to calculate the mean and variance of the soil pressures using Equations (5-4) and (5-6), respectively.

5.2 Site-Specific Geotechnical and Seismic Design Parameters

The design of the BWRX-300 is based on site-specific geotechnical inputs developed based on the results of site investigations and laboratory testing programs described in Section 3.1. Equivalent linear properties of soil and rock subgrade materials are developed for use as input for the static SSI analyses as described in Section 5.2.1.

Spectra defining the magnitude and frequency content of the site-specific design ground motion are developed, as described in Section 5.2.2, based on the results of site investigations and laboratory testing as well as the results of the site Probabilistic Seismic Hazard Analysis (PSHA). Probabilistic SRA are performed, as described in Section 5.2.2, to accommodate the effects of overlying materials in a manner which propagates the epistemic uncertainties and aleatory variabilities in the site parameters to preserve the desired hazard levels and performance goals. As described in Section 5.2.4, the results of these probabilistic SRA also serve as input for the development of stiffness and damping properties of subgrade materials that are compatible to the free-field strains generated by a typical design earthquake event. Five sets of ground motion time histories compatible to the ground motion design spectra are developed, as described in Section 5.2.3, for use as input for the linear seismic SSI analysis.

5.2.1 Equivalent Linear Subgrade Static Properties

The static earth pressure demands on the below grade exterior walls are obtained from 1-g static analysis of 3-D RB FE model embedded in a layered half-space continuum model representing the surrounding soil and rock. The following equivalent linear elastic properties are assigned to the solid FE of the half-space continuum model:

- Effective unit weight that for soil materials below groundwater table represents the total weight of soil minus unit weight of water;
- Elastic or Young's Modulus (E_{st}) representing linearized stiffness properties of the soil and rock for long-term static loading conditions; and
- Poisson ratio (ν_{st}) representative of at-rest lateral pressure conditions.

The design demands due to groundwater pressures is considered by a separate FE analysis where hydrostatic pressures are applied on the below-grade walls below the nominal groundwater level elevation.

As noted in Section 5.1.2, the weight of the rock materials can be neglected in the analysis if the rock mass is self-supporting, i.e., requires no lateral support when excavated. Section 5.1.3 presents an approach for including the design, the static and seismic lateral pressure demands from rocks that require stabilization. Effective unit weight properties are assigned to layers of weathered rock materials that require lateral support when excavated based on the results of non-linear FIA described in Section 4.0.

Effective unit weights of the subgrade materials for use as input for the static analysis are obtained from the results of site investigations and laboratory testing described in Section 3.1. Upper bound values for soil effective unit weight, which are calculated as mean plus one standard deviation of the measured values, are used as input for the analyses to address uncertainties in soil unit weight measurements.

Static analyses use E_{st} values representative of lower bound linearized subgrade stiffness properties that provide conservative upper bound subgrade deformation estimates under long-term static loading conditions. Values of E_{st} and ν_{st} for soil and rock mass layers are developed from data collected from site investigations and laboratory testing programs as described in Sections 5.2.4 and 5.2.1, respectively. Lower bound E_{st} values are obtained based on weighted log mean and log standard deviation of the measured values using appropriate weight factors reflecting the level of confidence on the data obtained from the different field and laboratory tests.

The profiles of equivalent linear subgrade properties for use as input for static analyses of the BWRX-300 RB are correlated with the results of non-linear soil stability analyses described in Section 3.2 to ensure the design envelopes all uncertainties related to non-linear behavior of soil and rock mass.

5.2.1.1 Equivalent Linear Stiffness Properties of Soil Materials

Values of E_{st} for soil materials for use as input for the static analyses are developed based on results of:

- field tests, such as cone penetration tests (CPT), standard penetration tests (SPT), pressuremeter and dilatometer tests; and
- triaxial unconsolidated undrained (UU) compression or the triaxial consolidated undrained (CU) compression laboratory tests of undisturbed specimens.

Appropriate correlation criteria are used to obtain estimates of the E_{st} for soil materials from the results of site investigations and laboratory testing. Lower bound values for the soil materials E_{st} can be obtained by:

- Reduction of the low-strain shear moduli obtained from V_S measurements
- Correlations with shear strength parameters such as undrained shear strength
- Correlation of stress-strain relationship measurements obtained from pressuremeter tests
- Correlations with N values, according to SPT resistance, drilling equipment energy measurements, and types of soil
- Correlations with CPT tip and skin resistance, which are numerous and related to the type of soil

Profiles of E_{st} may be developed based on the base-case profiles of small-strain (measured) shear velocities (V_S) used for the development of profiles of subgrade dynamic properties described in Section 5.2.4. The following equation may be used to relate the E_{st} values to:

$$E_{st} = D_E \, 2(1 + \nu_{st}) \, V_S^2 \, \rho \tag{5-11}$$

where:

D_E	is the monotonic stiffness degradation coefficient
$\rho = w/g$	is the soil mass density
w	is the dry soil unit weight
$g = 9.81\text{m/sec}$	is the Earth's gravity constant.

The D_E value can be based on monotonic elastic modulus (E/E0) degradation curves, such as the degradation curve on Figure 8-15 of FHWA NHI-16-072 (Reference 8.34), considering anticipated strain levels under long term static loads. Adequate lower bound E_{st} values are developed considering the mean and standard deviation values for V_S and D_E to account for uncertainties and variations of subgrade properties.

Data obtained from triaxial CU or UU compression tests of undisturbed clay specimens may be used to calculate E_{st}. Measurements of stress-strain relationships may be used directly to estimate E_{st} of clays. More reliable values often can be obtained from the laboratory test measurements of undrained shear strength (s_u) using empirical correlations such as the following empirical equation recommended in Reference 8.35:

$$E_{st} = \left(4200 - 142.5 I_p + 1.73 I_p^2 - 0.0071 \, I_p^3\right) s_u \tag{5-12}$$

where: I_p is the Plasticity Index.

Empirical correlations are also used to estimate E_{st} of soil layers from field test results. The following equation was proposed by Menard and Rousseau (Reference 8.36) to estimate equivalent-linear E_{st} of soil layers from pressuremeter field tests:

$$E_{st} = \frac{E_M}{\alpha} \tag{5-13}$$

where: E_M is the Menard's modulus calculated directly from the pressuremeter field measurements of soils under drained conditions; and

α Menard's correction factor.

Menard's α factor is applied to correct the E_M that usually underestimate the stiffness of the soil because it is developed from stress-strain measurements over a large range of strains assuming infinite borehole and uniform soil properties that remain undisturbed by the testing probe. Menard's α factor are determined empirically for different soil types and range from 0.25 to 1 according to Reference 8.37.

Table 5-2 provides examples of empirical correlations published in the literature for calculations of E_{st} of different types of soil materials from SPT and CPT results.

The following theory of elasticity equation is used to calculate ν_{st} values representative of soil at-rest (K_0) lateral pressure conditions:

$$\upsilon_{st} = \frac{K_0}{1 + K_0} \quad (5\text{-}14)$$

The BWRX-300 design considers upper bound values for at-rest coefficient K_0 to address uncertainties and variations of subgrade properties. The K_0 values are determined based on the results of site investigations and laboratory testing programs described in Section 5.2.4. Using measurements of effective angle of friction (ϕ_s), K_0 values for normally consolidated soils may be determined from the following simplified Jacky's equation:

$$K_0 = 1 - sin\ (\ \phi_s) \quad (5\text{-}15)$$

K_0 values for over-consolidated materials (e.g. stiff to hard clays) may be determined from the following modified Jacky's equation:

$$K_0 = [1 - sin\ (\ \phi_s)]\ OCR^{sin\ (\ \phi_s)} \quad (5\text{-}16)$$

where *OCR* is the over-consolidation ratio.

5.2.1.2 Rock Mass Equivalent Linear Properties

Equivalent linear E_{st} of rock masses can be estimated based on the intact rock Young's Modulus (E_{ri}) and the rock mass classification determined from results of the site investigation program. The following Hoek and Diederichs (Reference 8.23) equation may be used to adjust the intact rock E_{ri} and calculate rock mass E_{st} based on the rock mass Geotechnical Strength Index (GSI):

$$E_{st} = E_{ri} \left[0.02 + \frac{1 - 0.5D}{1 + e^{\left(\frac{60 + 15D - GSI}{11}\right)}} \right] (\text{GPa}) \quad (5\text{-}17)$$

where: E_{ri} and E_{st} are in units of giga Pascals (GPa); and

D is the degree of rock disturbance which values range from 0 for undisturbed confined rock to 1 for blast damaged rock in a typical open pit mine slope.

The following equation from Reference 8.21, which was developed by Galera, Alvarez, and Bieniawski, may be used to estimate rock mass E_{st} by adjusting the measured intact rock E_{ri} using its Rock Mass Rating (RMR) qualification:

$$E_{st} = E_{ri}\, e^{\left(\frac{RMR-100}{36}\right)} \text{ (GPa)} \tag{5-18}$$

where: E_{ri} and E_{st} are in units of GPa.

Results of UC strength laboratory tests performed on intact rock specimens can serve as the basis for development of E_{ri} values. Reliable, measured values of E_{ri} are often difficult to obtain due to sample damage from micro-cracking in recovered rock samples. The strength measurements obtained from UC strength tests are often considered more reliable because the sample damage has a greater effect on E_{ri} than on the UC strength. More reliable values of E_{ri} for use in Equations (5-17) and (5-18) can be obtained from the UC strength measurements as follows:

$$E_{ri} = MR \text{ (UC strength)} \tag{5-19}$$

where: MR are modulus ratio values like those provided in Table 3 of Reference 8.23 for various rock types and textures.

If UC strength measurements of intact rock E_{ri} are not available, the following equation proposed by Hoek and Diederichs in Reference 8.23 may be used to estimate E_{st} of the rock mass in GPa based solely on its GSI:

$$E_{st} = 100\left[\frac{1-0.5D}{1+e^{\left(\frac{75+25D-GSI}{11}\right)}}\right] \text{(GPa)} \tag{5-20}$$

where: D is the same rock disturbance parameter as the one used in Equation (5-17).

Empirical equations may be used to estimate E_{st} of the rock mass in GPa based on its RMR qualification. The following equation proposed by Serafim and Pereira in Reference 8.38 may be used to calculate rock mass E_{st} for values of RMR < 50:

$$E_{st} = \left[10^{\left(\frac{RMR-10}{40}\right)}\right] \text{ (GPa)} \tag{5-21}$$

The following equation proposed by Bieniawski in Reference 8.11 may be used to calculate rock mass E_{st} for values of RMR > 50:

$$E_{st} = [2(RMR) - 100] \text{ (GPa)} \tag{5-22}$$

Upper bound ν_{st} values for rock masses may be developed based on V_P and V_P measurements and the level of rock fracturing. It is anticipated the ν_{st} values developed based on V_S and V_P measurements will typically be higher than or similar to measurements on recovered rock samples due to the rock sample damage. For most rock masses, ν_{st} value is between 0.10 and 0.35. Lower ν_{st} values are associated with highly fractured rock masses, and higher ν_{st} values with intact rock masses.

Equivalent linear rock stiffness properties may further be adjusted based on the results of non-linear FIA as described in Section 5.1.3.

Table 5-2: Correlations for Estimation of Soil Young's Modulus from SPT and CPT

Soil Type	Soil Static Equivalent Linear Modulus (E_{st}) in (kPa) from	
	SPT Measured N-values	CPT Measured q_c-values
Sands (normally consolidated[1]	$E_{st} = 500\ (N + 15)$ $E_{st} = 7000\sqrt{N}$ $E_{st} = (15000\ \ to\ 2000)\ \ln\ (N)$	$E_{st} = 8000\sqrt{q_c}$
Saturated Sand[1]	$E_{st} = 250\ (N + 15)$	$E_{st} = 5.2 q_c$
Sands[1] (Normally Consolidated)	$E_{st} = (2600\ to\ 2900)\ N$	$E_{st} = (6\ to\ 30)\ q_c$
Sands[2] (Normally Consolidated)	$E_{st} = (194 + 8N)(1 - \nu^2)$	$E'_{st} = (2\ to\ 40)\ q_c$ [3]
Sand[1] (Over-consolidated)	$E_{st} = 40000 + 1050\ N$	$E_{st} = (6\ to\ 30)\ q_c$
Sand[2] (Preloaded)	$E_{st} = (420 + 10N)(1 - \nu^2)$	
Gravelly Sand[1]	$E_{st} = 1200\ (N + 6)$ or $E_{st} = 600\ (N + 6)$ for $N \leq 15$ $E_{st} = 600\ (N + 6) + 2000$ for $N > 15$	N/A
Clayey Sand[1]	$E_{st} = 320\ (N + 15)$	$E_{st} = (3\ to\ 6)\ q_c$
Silts[1][3]	$E_{st} = 320\ (N + 15)$ $E_{st} = 300\ (N + 6)$	$E_{st} = 2.5\ q_c$ for $q_c < 2500\ kPA$ $E_{st} = 4q_c + 5000$ for $q_c \geq 2500\ kPA$
Soft Clays[1]		$E_{st} = (3\ to\ 8)\ q_c$
Sands, Silts, and Clays[2]		$E'_{st} = \alpha_c \cdot q_c$ α_c Is function of soil type and q_c

Table 5-2: Correlations for Estimation of Soil Young's Modulus from SPT and CPT

Soil Type	Soil Static Equivalent Linear Modulus (E_{st}) in (kPa) from	
	SPT Measured N-values	CPT Measured q_c-values
Clays(1)(4)(5)	$E_{st} = 12 \cdot k \cdot N$ for plastic clay CH $E_{st} = 6 \cdot k \cdot N$ for lean clay CL $E_{st} = 3 \cdot k \cdot N$ for sandy clay (CS) and silt (ML)	

NOTES:

(1) from several literature references compiled in Reference 8.35

(2) Compiled in ASCE Technical Engineering and Design Guide No.9 (Reference 8.39)

(3) E'_{st} is constrained modulus $E_{st} = E'_{st}\frac{(1+\nu)(1-2\nu)}{(1-\nu)}$

(4) k is a function of soil type and the clay Plastic Index I_p

(5) Terzaghi, Peck, Sowers, compiled in Reference 8.31

5.2.2 Development of Site-Specific Ground Motion Spectra

The development of SSE ground motion for the seismic design of SC-I SSCs begins with the PSHA that defines the reference site hazard by horizontal and vertical Uniform Hazard Response Spectra (UHRS). Per RG 1.208 guidance, the UHRS are calculated for annual exceedance frequencies (AEFs) of 10^{-4}, 10^{-5} and 10^{-6} year^{-1}. Ground Motion Prediction Equations (GMPEs) are used appropriate for a defined reference site condition such as hard base-rock defined by V_S and associated shallow crustal damping parameter, kappa. This reference site hazard, as defined, is appropriate for any location within the BWRX-300 plant area.

To develop the design ground motion for the seismic analysis of BWRX-300 structures, the reference site hazard must be adjusted to accommodate the effects of overlying materials in a manner which preserves the desired hazard levels and performance goals. Probabilistic SRAs are performed to calculate site amplifications used for development of horizontal UHRS defining the seismic hazard at elevations above the elevation of the reference site seismic hazard condition. Approach 3, defined in NUREG/CR-6728 (Reference 8.40), is implemented by adjusting the reference site hazard condition for overlying materials to properly accommodate the following two types of variability in dynamic subgrade material properties:

- The aleatory variability that is due to the natural randomness or fluctuations of dynamic properties of subgrade in-situ materials with depth and across and around the structural footprint that is introduced in the SRA based on generic empirical probability distributions

as random variations about base case V_S values and base-case shear modulus degradation and hysteretic damping curves defining the dependence of material stiffness and damping properties with strain; and

- The epistemic uncertainty that arises from incomplete knowledge of the dynamic properties of the subgrade materials that is usually addressed in the probabilistic SRA by consideration of multiple base-case models of subgrade properties and associated weight factors.

The base-case models that are developed based on results of geotechnical site investigation and laboratory test programs conducted per recommendations in Section 3.1, are expressed in terms of estimates of median values and standard deviations. Per regulatory guidance of SRP 3.7.1 and provisions of ASCE/SEI 4-16 (Reference 8.7), Section 2.3.1, base-case profiles are defined as horizontal layers with specified thickness and values of soil unit weight (γ) small-strain V_S and Poisson ratio (μ). These base-case subgrade profiles reflect the as-built site conditions at the site and account for removal of surficial materials. Base-case degradation curves define the variations of dynamic shear modulus (G) and hysteric damping (β) properties of the different subgrade materials as a function of the soil strain are used to represent the nonlinear behavior of the modeled subsurface materials.

In accordance with SRP 3.7.1 Subsection II.4.A.iv guidance and provisions of ASCE/SEI 4-16, Section 2.3.2.1, the base-case models are randomized into 60 realizations about their respective base-case values based on the assigned aleatory standard deviation values. Equivalent-linear one-dimensional (1-D) wave propagation analysis based on the SHAKE methodology are performed for each combination of randomized base-case V_S profiles and base-case shear modulus degradation and hysteretic damping curves. The reference site control motions for these probabilistic SRA may be generated with the point-source band-limited white noise models. For sites where region-specific source spectral shapes differ significantly from the point-source model, control motions for the probabilistic SRA may be generated by spectral matching or Random Vibration Theory (RVT) to match the response spectral shapes generated from the reference site UHRS.

The backfill that may be placed to form the plant grade may be included in the base-case models if their horizontal extent is large enough to justify the 1-D SRA assumption of infinite horizontal layering. The dynamic properties of backfill materials and related uncertainties may be assumed based on generic properties of similar materials and confirmed later by testing when the source of backfill material is confirmed.

The probabilistic SRA using Approach 3 as defined in NUREG/CR-6728 (Reference 8.40), provide amplification factors which are integrated with the reference rock hazard curves to produce site-specific hazard curves and UHRS defining the seismic hazard in the horizontal direction at the FIRS (foundation bottom) elevation, the PBSRS (profile surface) elevation and the PBIRS (at intermediate embedment depth) elevations.

The intermediate elevations where PBIRS are calculated, are selected based on the features of the V_S basecase profiles used for the probabilistic SRA. The elevations corresponding to significant V_S contrasts in the base-case subgrade profiles are included as intermediate elevations. The central elevation between the profile surface and the foundation bottom may be used for base-case profiles

with a relatively uniform variation of V_S with depth (Reference 8.41). One or more additional elevations between the profile surface (i.e. PBSRS) and the foundation bottom (i.e. FIRS) are included when the base-case profile includes nonuniform variations such as significant velocity inversions from subsurface conditions such as strong velocity increases at a rock-to-soil transition or low-velocity zones. The intermediate elevations (i.e. PBIRS) corresponding to these significant velocity inversions are included to verify the adequacy of the SSI profiles (Reference 8.41).

Following the performance-based approach specified in ASCE/SEI 43-05 (Reference 8.4), Section 2.1, horizontal FIRS, PBSRS and PBIRS at 5% damping are developed using the corresponding UHRS with an AEF of 10^{-4} and 10^{-5} yr^{-1}.

Frequency-dependent Vertical/Horizontal (V/H) ratios are used to define the vertical component of the site-specific design motion. The vertical FIRS, PBSRS, and PBIRS are developed either:

- directly by applying the V/H ratios to the horizontal FIRS, PBSRS, and PBIRS; or
- from vertical UHRS calculated by Approach 3 integration of the V/H ratio with the horizontal site-specific hazard curves.

The V/H ratios are defined based on NUREG/CR-6728 (Reference 8.40) or more recent models, such as those provided in Reference 8.42 and Reference 8.43, according to the type of subgrade conditions at the site that is typically defined by the average shear wave velocity (V_{S30}) of the top 30 m of subgrade materials. If the site is characterized by variations in subgrade conditions with depth, following the guidance of EPRI 3002011804 (Reference 8.44), different V/H ratios may be applied to the horizontal FIRS, PBSRS, and PBIRS to calculate the corresponding vertical ground motion design spectra.

Per 10 CFR 50, Appendix S, the BWRX-300 seismic design ensures that all SC-I SSCs can resist a minimum level of ground motion irrespective of the site-specific hazard results. To meet this regulatory requirement, the SSE spectrum defining the horizontal free field ground motion for use as input for the SSI analyses is checked to ensure it envelopes the minimum horizontal earthquake response spectrum. The minimum level of free field horizontal ground motion is defined by minimum 5% damped SSE response spectrum with a piece-wise linear spectral shape of 5% damped generic spectra on Figure 1 of RG 1.60 anchored at a peak ground acceleration (PGA) of 0.1 g.

5.2.3 Development of Ground Motion Acceleration Time Histories

Acceleration, velocity, and displacement time histories of the outcrop design ground motion are developed by fitting appropriately selected sets of recorded seed time histories to 5% damped ground motion SSE spectra following the regulatory guidance of SRP 3.7.1 and requirements of ASCE/SEI 43-05 (Reference 8.4), Section 2.4. Seed time histories shall be selected from an appropriate database of recorded time histories (e.g. NUREG/CR-6728, Reference 8.40) that have spectral shapes reasonably consistent with the spectral shape of the design target spectrum over the frequency range of interest and characteristics that reasonably represent the earthquake motions expected at the site.

Per requirements of ASCE/SEI 4-16 (Reference 8.7), Section 2.6.1, at least five sets of acceleration time histories (ATHs) are developed for the linear elastic seismic analysis to mitigate

the uncertainty in the computed responses due to the phasing of the time history frequency components.

The spectral matching procedure is implemented for fitting the seed time histories to the 5% damped target spectra that retains the phase spectra of the seed time histories, preserving the relative phasing between horizontal and vertical components, as well as, preserving the non-stationarity and randomness characteristics. The modified time histories are checked as follows to ensure they meet the criteria specified in ASCE/SEI 43-5, Section 2.4 (Reference 8.4):

1. The 5% damped Acceleration Response Spectra (ARS) of the modified seed time history shall be computed at a minimum of 100 points per frequency decade, uniformly spaced over the log frequency scale. The average of 5% damped ARS of the five ATHs shall be compared to the 5% damped target acceleration spectrum at each frequency point in the range of 0.1 Hz to 100 Hz to ensure that:
   a. the average ARS does not fall below the target spectra by more than 10% at any frequency point; and
   b. the average ARS does not fall below the target spectra at no more than nine adjacent frequency points.
2. If the acceleration spectrum for the modified ground motion histories exceeds the target spectrum by more than 30% at any frequency between 0.2 Hz and 25 Hz, the power spectral density of the modified ground motion history shall be computed as described in ASCE/SEI 4-16, Section 2.6.2, and shown not to have significant gaps in energy at any frequency over this frequency range.
3. The total duration of time histories shall be long enough to provide an adequate representation of the Fourier components at low frequency.
4. In general, time histories used as input for the seismic response analyses should have a strong motion duration, and ratios V/A and AD/V^2 (where A, V, and D are the peak ground acceleration, velocity, and ground displacement, respectively) that are consistent with those of appropriate controlling events considered in the PSHA.
5. The three modified ATHs representing the ground motion in the three orthogonal directions (two horizontal and one vertical) shall be statistically independent. Each pair of ground motion histories is considered statistically independent when the absolute value of their correlation coefficient does not exceed 0.16.
6. The ATHs shall be baseline corrected to ensure the ground velocity converges to zero at the end of the earthquake record and maintains a zero-mean value over the time history duration.

The time step of the time histories is 0.005 sec or less. The time step of the modified time histories may be refined by zero padding in the Fourier spectra frequency domain to ensure they will retain the frequency content of the original input motion. For hard rock high frequency (HRHF) sites, the time step of the time histories used for the calculation of in-structure response spectra (ISRS) for design and evaluation of important to safety SSCs is refined to 0.0025 sec to ensure the accuracy of ISRS up to 50 Hz per requirements of DC/COL ISG-01 (Reference 8.8), Section 3.1.1. The time step of 0.0025 sec is less than a sixth (1/6) of the Nyquist Frequency of 50 Hz, which, as

described in Reference 8.45, ensures that the related errors in the calculated ISRS for frequencies up to 50 Hz remain below the error criteria of 10% set by ASCE/SEI 4-16 (Reference 8.7).

The ground motion design spectra developed as described in Section 5.2.2 and the ATHs fitted to these spectra represent the outcrop motion. The outcrop ground motion ATHs developed by fitting ground motion records to the PBSRS can be directly used as input for the SSI analyses because the outcrop and in-layer motion at the surface of the profile are identical. The FIRS and PBIRS compatible outcrop motion ATHs are converted to incoming or in-layer motion ATHs for use as input for the frequency-domain SSI analyses. To convert the outcrop ATHs to in-column ATHs, time domain linear elastic site-response analyses are performed on the subgrade profiles of strain-compatible V_S and V_P used for the SSI analysis, developed as described in Section 5.2.4.

5.2.4 Strain Compatible Subgrade Dynamic Properties

In accordance with NUREG-0800, SRP 3.7.2 Subsection II.4 guidance and the requirements of ASCE/SEI 4-16 (Reference 8.7), Section 5.1.4, the deterministic SSI analyses are performed for a set of subgrade properties to address:

- aleatory variations of subgrade conditions across and around the footprint of the foundation;
- epistemic uncertainties related to the determination of subgrade dynamic properties; and
- effects of primary non-linearity of subgrade materials induced by the free-field excitation.

To address the uncertainties related to the variations and determination of subgrade conditions, a set of at least three seismic SSI analyses are performed using the Best Estimate (BE), Lower Bound (LB), and Upper Bound (UB) subgrade profiles reflecting the as-built site conditions at the site. After the results obtained from the analysis of each subgrade profile for the five sets of ATHs are averaged as described in Section 5.2.3 to address uncertainty due to the phasing of the time history frequency components, these averaged results of the SSI analysis of different subgrade profiles are enveloped to account for the variations and uncertainties in the determination of input subgrade properties. The effects of primary non-linearity of subgrade materials response are addressed by using dynamic stiffness and damping properties which are compatible to the free-field strains induced by an SSE level seismic event.

BE, LB and UB profiles are developed that represent the variations of strain-compatible V_S and damping as well as the density of far-field and near-field subgrade materials as function of depth. The BE, LB, and UB V_P subgrade properties are calculated from the corresponding strain-compatible V_S properties using the following elastic theory equation:

$$V_P = V_S \sqrt{\frac{2(1-\upsilon)}{1-2\upsilon}} \tag{5-23}$$

where: υ are the mean values of the subgrade material small strain Poisson ratio.

The V_P of the softer subgrade materials located below groundwater level is adjusted to values close to the water Vp ≈1,450 m/s to account for the presence of groundwater as required by ASCE/SEI 4-16 (Reference 8.7), Section 5.2(d). The adjusted Poisson ratio values are checked to

ensure the values are kept below the 0.48 needed for the numerical stability of the SSI analysis results.

The UB SSI analysis profiles include the higher V_S and V_P values in conjunction with the lower damping values. The LB SSI analysis profiles include lower V_S and V_P values in conjunction with higher damping values. The mean value for the density of subgrade materials is assigned to all three profiles.

Section 5.1.7(c) of ASCE/SEI 4-16 requires the properties used as input for the SSI analyses to be consistent with the soil properties used in the generation of input motion. The results of probabilistic SRA described in Section 5.2.2 are used for development of SSI analysis profiles of strain-compatible dynamic subgrade properties which are consistent with the probabilistically based design motions. Strain-compatible subgrade properties are developed that reflect both the hazard level (ground motion and AEF) as well as the aleatory variabilities and epistemic uncertainties of site-specific dynamic material properties incorporated in developing the design motions.

As noted in NUREG/CR-6728 (Reference 8.40) and RG 1.208, simply using control motions based on a generic rock site hazard to drive the site-specific soil column may not result in strain-compatible properties consistent with the site-specific hazard developed using a fully probabilistic approach. Therefore, an approach is used to develop Hazard Consistent Strain-Compatible Properties (HCSCP) which are consistent with the site-specific probabilistic hazard. This new approach assumes strain-compatible properties are approximately lognormally distributed, consistent with observed strong ground motion parameters (Reference 8.46) and makes use of the distributions of strain-compatible properties catalogued during development of the suites of amplification factors.

The site-specific horizontal hazard curves at the AEF of interest are examined to determine the ground motion levels (interpolating logarithmically as necessary) and locate the corresponding amplification factors and associated strain-compatible V_S and damping properties at the ground motion levels determined from the hazard curve. For each case of epistemic variability (i), median (μ_i) and standard deviation (σ_i) estimates (over aleatory variability) are interpolated (logarithmically) to the appropriate ground motion as specified by the site-specific hazard curve at the desired annual exceedance probability. To accommodate epistemic variability in the subgrade properties, the same weights (w_i) that are used in developing the site-specific hazard curves are applied to the corresponding strain-compatible properties. The weighted median (mean log) set of strain-compatible properties (for each layer) are calculated, as explained below, while the associated variance includes both the aleatory component for each epistemic case as well as the variability of mean properties for each base-case.

To examine consistency in HCSCP across structural frequency, as the magnitude contributions can vary, the entire process is performed at PGA (frequency of 100 Hz) and at low frequency of 1 Hz. Because amplification factors are typically developed for a range in magnitude reflecting contributions at lower (≤ 2 Hz) frequency range and higher (≥ 2 Hz) frequency range, the consistency check at PGA and 1 Hz covers the typical range in control motions.

The properties are interpolated to the desired PGA and 1.0 Hz levels for each case of epistemic uncertainty (i) with assigned weight factor (w_i) and having log-median (μ_{ln_i}) and log-standard

deviation (σ_{ln_i}) properties. Each case of epistemic uncertainty is then combined as follows in weighted median properties (μ_{ln}):

$$\mu_{ln} = \sum_i w_i \mu_{ln_i} \tag{5-24}$$

as well as weighted log-normal variances ($\mathrm{Var}_{(ln)}$) that include the site epistemic uncertainty (different medians) in the combined properties:

$$\mathrm{Var}_{(ln)} = \sum_i \left[w_i \sigma_{ln_i}^2 + w_i \left(\mu_{ln_i} - \mu_{ln} \right)^2 \right] \tag{5-25}$$

The weighted average (μ_{ln}) values for the strain-compatible V_S and damping subgrade properties calculated from Equation (5-24) are adopted as the BE properties for site-specific SSI analysis. The variations $\mathrm{Var}_{(ln)}$ obtained from Equation (5-25) for PGA and 1 Hz levels are used to define the LB and UB strain-compatible V_S and damping subgrade properties (note that LB and UB are different than LR and UR used as base-case profiles, respectively).

The calculated LB and UB V_S properties are then checked to ensure that the following minimum variations requirements of Section 5.1.7(d) of ASCE/SEI 4-16 (Reference 8.7) are met:

- UB V_S values calculated as $\mu_{ln} + \mathrm{Var}_{(ln)}$ are at least $\sqrt{1.5}$ times larger than the BE V_S values, and
- LB V_S values calculated as $\mu_{ln} - \mathrm{Var}_{(ln)}$ are at least $\sqrt{2/3}$ times smaller than the BE V_S values.

In accordance with the recommendations of ASCE/SEI 4-16, Section C5.2, the soil damping values used as input to the SSI analysis may be limited to a maximum of 2% for very low ($\leq 10^{-4}$%) strains and to maximum of 15% at large strains.

5.3 Reactor Building Seismic Soil-Structure Interaction Analysis

Seismic demands for design of the BWRX-300 RB SSCs are obtained from SSI analyses performed following SRP 3.7.2 guidelines and the requirements of Section 5 of ASCE\SEI 4-16 (Reference 8.7).

The seismic SSI analyses are performed using the sub-structuring method, as described in Sections 5.4 and C5.4 of ASCE\SEI 4-16, and the SASSI (a system for analyses of soil-structure interaction) analysis approach to calculate, the seismic response of SSI system consisting of the RB structure, the surrounding subgrade and the excavated volume of the subgrade materials replaced by the embedded portion of RB structure, backfill materials and/or excavation support structures. The sub-structuring allows the seismic response of this linear SSI system to be obtained by subdividing the problem into a series of simple subproblems that can be solved separately. Using the principle of superposition, the results of different sub-analyses are combined to obtain the final solution for the SSI problem. Linear-elastic SASSI analyses are performed in the frequency domain for a set of frequencies selected as described in Section 5.3.2.

As described in Section 5.1.2, models are used for the SSI analysis that assume isotropic elastic material properties of structural members and surrounding subgrade and neglect any non-linearity at the soil-structure contact interfaces. Linear-elastic material constitutive models are based on complex moduli, which produce frequency-independent hysteresis damping. This allows damping to be assigned to the SSI model that adequately represent the damping properties of different

materials. For each frequency of analysis, a complex impedance matrix is calculated that defines the force-displacements relationship at each interaction node.

The SASSI analyses are performed on one-step structural models that accurately represent the geometry and dynamic properties of the RB structure and its interaction with the subgrade. These structural models have a refined FE mesh that is identical to the mesh of the models used for the static analyses. The dynamic properties of subsystems, components, and equipment are included in the RB FE model as described in Section 5.3.6 that also presents an approach for addressing the effects of Equipment-Structure Interaction (ESI) in the seismic response analysis.

The linear-elastic assumption eliminates the need for defining initial conditions and allows a set of design and sensitivity SASSI one-step approach analyses to be performed on refined RB structural models with a large number of interaction nodes. The superposition principle, which is applicable only for linear elastic analyses, allows the SASSI stress results obtained from different dynamic and static analyses to be combined with the results of static analyses in seismic design load combinations.

The SASSI extended subtraction method (ESM) simplification may be used for calculations of the SSI system impedance matrix, where only a selected set of nodes of the excavated volume are specified as interaction nodes. Interaction nodes are established in the ESM model at:

- interfaces between the excavated volume and structural models;
- the excavated volume top surface located at the PBSRS elevation; and
- planes within the excavated volume located at PBIRS elevations.

Additional interaction nodes may be included in layers of softer soil material to improve the accuracy of the SSI solution. The accuracy of the solutions obtained from the ESM analyses is demonstrated based on the guidelines provided in SRP 3.7.2. The validation of ESM may be based on comparisons of results obtained from the analyses of reduced (quarter or half) size models performed using the ESM and the SASSI flexible volume or direct method (DM), where all nodes of the excavated volume are specified as interaction nodes.

Far-field interaction nodes are also established at the surface of each soil layer through the RB shaft embedment depth. These interaction nodes that are located at least 50,000 ft away from the FE model are used to capture the horizontal and vertical components of the far-field motion in the SSI model. The responses calculated from these far-field interaction nodes are used to monitor the propagation of the input control motion through the RB embedment depth.

The SASSI site response solution defining the free-field displacement amplitudes at the interaction node locations is obtained using subgrade models that are consistent with the SRA models, described in Section 6.2.2, used for development of design ground motion spectra. To account for the non-linear response of subgrade materials, strain-compatible subgrade properties are used that are developed, as described in Section 5.2.4, based on the results of equivalent linear probabilistic SRA. The uncertainties related to variation of soil and rock properties are addressed by using seismic demands for the design of RB SSCs that are obtained as envelope the results obtained from SSI analysis cases performed using a set of at least three subgrade profiles representing BE, LB, and UB properties of subgrade materials.

Input ground motion ATHs, which are developed as described in Section 5.2.3, are applied to SASSI models as vertically propagating coherent:

- shear waves for horizontal components of the input motion; and
- compression waves for the vertical component of the input motion.

The horizontal control motion is applied to the SASSI model in a manner that is consistent with the 1-D wave propagation SRA approach used to account for the wave propagation characteristics of the site when defining the design ground motion spectra.

The effects of non-vertically propagating shear waves on the seismic response and design of RB SSCs are addressed as described in Section 5.3.3.

The effects of ground motion incoherency on the BWRX-300 seismic design is conservatively neglected. For HRHF sites, the effects of ground motion incoherency may be included in the design by using coherency function specified in Section 2.0 of DC/COL ISG-01 (Reference 8.8) or other coherency functions adequate for the site-specific conditions. If the ground motion incoherency effects are included in the design, the evaluation needs to include:

- comparisons of the coherent and incoherent responses and demands, and
- consideration of the potential variation of the coherency function with depth.

If the effects of ground motion incoherency are considered, the BWRX-300 seismic design will:

- include potential increases of rocking and torsional responses;
- not consider reduction in structural load demands; and
- limit the ISRS reductions in accordance with the regulatory guidance of SRP 3.7.2.

In accordance with ASCE 4 (Reference 8.7), Section 2.6.1, at least five sets of input motion ATHs are used as input for the SSI analyses to mitigate the uncertainty in the computed responses due to the phasing of the time history frequency components.

In accordance with the regulatory guidance of DC/COL ISG-017 (Reference 8.6), Section 5.2.3, the input ground motion is applied to the SSI model at foundation bottom elevation. Procedures described in Section 5.3.4 are used to ensure the ground motions used for the deterministic SASSI analysis are hazard consistent with the results of probabilistic site response analyses described in Section 5.2.

A set of SASSI analyses are performed to address the uncertainties related to variations of important SSI parameters and the simplified modeling assumptions in accordance with the regulatory guidance of SRP 3.7.2 and the requirements of ASCE/SEI 4-16 (Reference 8.7), Section 5.1. The results of SSI analyses performed using different subgrade profiles are enveloped to account for the variations and uncertainties in the determination of input subgrade properties.

Uncertainties related to variations in structural material properties including concrete cracking are addressed as described in Section 5.3.5. As described in Section 5.3.6, simple models representing the dynamic properties of the structures and foundations surrounding the RB are also included in the FE model to capture the SSSI effects .

Sensitivity SSI analyses are performed to address the uncertainties related to effects of:

- the excavation support and fill concrete as described in Section 5.3.8;
- the soil separation as described in Section 5.3.9; and
- groundwater level variations as described in Section 5.3.10

These sensitivity analyses are performed on SSI models representing conditions that bound the variation of these effects. Results of these sensitivity analysis are compared with results of design basis SSI analyses to evaluate the importance of these effects on the RB seismic response and design. These comparisons are performed for key response parameters, selected as described in Section 5.3.1.

If the comparisons indicate that the effects can result in responses that are significantly (>10%) higher than the responses obtained from the design basis analyses, the results of the sensitivity analysis are incorporated in the BWRX-300 seismic design basis to ensure that the seismic design bounds these uncertainties.

If the site is characterized by a high seismicity and the results of non-linear static FIA, described in Section 4.0, indicate that the non-linear response of subgrade materials is significant, seismic SSI analyses are performed on non-linear models, as described in Section 5.3.11, to assess explicitly the following non-linear effects on the RB seismic response and design:

- Secondary non-linearity of subgrade materials
- Non-linearity at soil-structure interfaces such as separation and sliding
- Non-linearity at rock discontinuities.

5.3.1 Key Seismic Responses

A set of SSI analyses are performed to address the effects of variations of SSI parameters by comparisons of responses at key nodal locations. These key locations are selected based on the following criteria:

- Nodes at intersections of main structural members (main structural walls) at ground and other major floor elevations to illustrate global responses that exclude possible local effects due to out-of-plane vibrations of slabs and walls, openings or connections with columns, beams or subsystem supports.
- At least two roof nodes, one central and one corner node, to show all important modes of seismic response of structure including the effects of rocking and torsion.
- At least two basemat nodes, one central and one corner node, to show the SSI effects on the translational as well as the rotational (rocking and torsion) responses of foundation.

The seismic demands on the below grade portion of the RB structure are affected by the deformations resulting from the response of the SSI system. Therefore, besides the in-structural responses, main stress demand components, such as in-plane shear force and vertical bending moment demands, are also compared to be able to gain a complete understanding of the effects of SSI parameters variations on the structural design. These comparisons are performed for the main below-grade structural members at selected design cross-sections subjected to high seismic stress demands.

5.3.2 Frequencies of Analysis

The solution for the response of the SSI system is obtained at a selected set of frequency points and then interpolated for other frequency points. The analysis is performed for a cut-off frequency values established based on the largest value required the following four criteria of ASCE/SEI 4-16 (Reference 8.7), Section 5.3.5(b):

1. Twice the highest dominant frequency of the coupled soil-structure system, or
2. The highest structural frequency of interest, or
3. The frequency at which the Fourier amplitude of input motion has passed its peak value and has reached 10% of the peak value, and
4. 20 Hz.

The highest dominant frequency set by criterion (1) above is determined for each SSI analysis case based on the acceleration transfer function results representing the in-structure responses at the key locations that are defined as described in Section 5.3.1. The acceleration transfer function amplitudes are plotted, and the highest SSI response frequencies are determined based on the frequencies corresponding to the dominant peaks in the transfer function results.

Lower cut-off frequency values based solely on Criterion (1) above may be used if it can be demonstrated by performing sensitivity analyses that establishes that:

- the seismic demands, calculated for key design cross-sections, selected as described in Section 5.3.1, are adequate for design of the structural members and no more than 10% change is expected if higher frequency cutoff is used;
- the calculated 5% damped ISRS at key locations, selected as described in Section 5.3.1, are adequate for design and qualification of components and equipment and no more than 10% change is expected if higher frequency cutoff is used.

These sensitivity SSI analyses are performed for the stiffest subgrade condition on structural models with upper bound structural stiffness properties that provide bounding responses at high frequencies.

The value of cut-off frequency determined by the criteria described above is used for the analysis of the stiffest subgrade profile and models with upper bound structural stiffness properties. The analyses of softer subgrade profiles or reduced structural stiffness properties may use lower values for the cut-off frequency. In this case, it shall be demonstrated that the analysis of the stiffest profile provides responses that are bounding for frequencies higher than the cut-off frequencies used for the analyses of the softer subgrade profiles by comparing transfer function and 5% damped ISRS results for responses at key locations within the building, selected as described in Section 5.3.1.

The frequencies of analysis are selected at sufficiently small frequency intervals. Transfer function amplitude results for responses at the key locations, selected as described in Section 5.3.1, are inspected to detect any numerical anomalies in the interpolated transfer functions (e.g., sharp narrow spikes) that can potentially affect the accuracy of results. If present, the effects of these anomalies in the interpolated transfer function results shall be evaluated using additional

frequencies of analysis to ensure the anomalies in the transfer function interpolations do not affect the accuracy of the calculated responses.

Acceleration transfer functions and 5% damped ARS are also calculated for the response of SSI model free-field interaction nodes to check the amplitude and frequency content of the in-column free-field motion throughout the RB embedment depth.

5.3.3 Effects of Non-Vertically Propagating Seismic Waves

Site-specific sensitivity evaluations are performed on the potential effects of non-vertically propagating seismic waves on the free-field ground motion and the SSI response of BWRX-300 RB structure.

In general, the sensitivity evaluations of potential effects of non-vertically propagating seismic waves on the BWRX-300 seismic design consider:

- the effects of multidimensional, two- or three-dimensional (2-D or 3-D), wave propagation resulting from site characteristics like dipping bedrock surfaces, dipping subgrade layers, topographic effects, and other impedance boundaries, and
- the effects of local seismic sources generating inclined waves.

5.3.3.1 Evaluations of Multidimensional Wave Propagation Effects

As described in Section 5.2.2, probabilistic 1-D SRA are performed to develop the FIRS, PBIRS, and PBSRS defining the amplitude and frequency content of the site-specific ground motion used for the BWRX-300 seismic design. The basic assumption of the 1-D SRA is that the subgrade consists of horizontally infinite horizontal layers in which the seismic input motion propagates vertically from the hard rock, where the site reference seismic hazard is defined, to the ground surface. Site characteristics like dipping bedrock surfaces, dipping subgrade layers, topographic effects, and impedance boundaries can affect the pattern of seismic wave propagation resulting in non-vertically propagating seismic waves.

As discussed in NUREG/CR-0693 (Reference 8.47), horizontal and vertical responses are not significantly affected by dipping bedrock surfaces of 30 degrees or less and dipping soil layers of 20 degrees or less when compared to 1-D wave propagation analyses using SSI models. Based on these results, a 1-D wave propagation analyses are considered appropriate for bedrock surfaces and soil layers dipping less than these limits.

Multidimensional, 2-D or 3-D, wave propagation sensitivity analyses may be required to study the potential generation of inclined seismic waves when site characteristics significantly deviate from the basic assumption of infinite horizontal layers. These deterministic sensitivity SRAs are typically performed on two models with the same subgrade material properties and configurations as the BE base-case profiles used for the 1-D SRAs described in Section 5.2.2. Two sets of deterministic SRAs are performed on models representing:

a) the base-case profile used for the probabilistic SRA that assumes idealized site conditions with infinite horizontal layers, and

b) the actual site characteristics including dipping bedrock surfaces, dipping subgrade layers, topographic effects, and impedance boundaries.

Control motions may be applied to these SRA models at the bedrock surface elevations where the site reference seismic hazard is defined. The amplitude and frequency content of the input control motions are selected based on the PSHA results for rock-based UHRS with AEFs of 10^{-4} and 10^{-5} year^{-1}. For the sites where the non-linearity of subgrade materials can have a significant effect on the site response, equivalent linear sensitivity SRA should be performed using two or more UHRS controlling earthquakes with energy contents that dominate appropriate frequency ranges. For example, two control motions may be used representative of a high-frequency earthquake that dominates at high frequency range (5 and 10 Hz) and a low-frequency earthquake that dominates at low frequency range (1 and 2.5 Hz). ATHs or RVT control motions may be used that match the spectral shapes generated from the reference site UHRS.

Site amplification factors are calculated based on the 5% damped ARS results of each deterministic SRA for the site response at the FIRS, PBSRS and PBIRS elevations. Comparisons are made of the amplification results obtained from the SRA of model representing 1-D and multidimensional site conditions to determine if the site characteristics increase, decrease, or produce similar site response results. Based on these comparisons, the FIRS, PBIRS, and PBSRS developed as described in Section 5.2.2 based on the results of 1-D probabilistic SRA analyses may be increased to address the effects of inclined wave propagation on the free-field site response.

5.3.3.2 Evaluations of Local Seismic Source Effects

The presence of a local seismic source may also generate inclined waves due to the potential source-to-site effects on the wavefield. Generally, the angle of incidence of the seismic waves decrease as the waves propagate towards the ground surface due to Snell's law. Thus, for non-uniform sites with softer soils in layers that create a vertical velocity gradient, the effects of inclined waves are reduced due to this decrease in the angle of incidence. NUREG/CR-6728 (Reference 8.40) indicates rock sites at distances from the source of about 10 to 15 kilometers or less show inclined shear wave motions. Substantial inclined shear wave motions are not shown for rock and soil sites that are at distances of more than 15 km from the source so for these sites, the local seismic source effects on the BWRX-300 seismic design can be neglected.

When a local seismic source is present near a more uniform site, one way to evaluate the effects of inclined waves from a local source is through modeling of the seismic wave propagation through the site as described in Reference 8.27. This model may include the source with a representation of the fault orientation, and the local geology to estimate the range of inclined wave angles that may affect the site.

NUREG/CR-6896 (Reference 8.48) presents a study of the effects of inclined seismic waves on deeply embedded structures in uniform and layered profiles. The study concluded that the SH waves representing the horizontal components of inclined shear waves have small effects on the SSI responses at the basemat elevation and that the SV waves representing the component of inclined shear waves polarized in the vertical plane, induce the highest peak response. Therefore, the effects of inclined SH waves on the BWRX-300 seismic design are neglected.

The results of NUREG/CR-6896 (Reference 8.48) study indicated that the SV waves may have effect on the SSI response at the basemat that is largest when the inclined angle, measured from the vertical axis to the direction of the inclined wave propagation, is near the critical angle of

incidence. The critical angle of incidence ($\emptyset_{cr}$) is a function of Poisson's ratio (υ) of the layer and is defined as following:

$$\emptyset_{cr} = \frac{\pi}{2} - \arctan\sqrt{\frac{1}{1-2\,\upsilon}} \qquad (5\text{-}26)$$

As described in Reference 8.49, for angles of incidence greater than the critical angle, the SV waves are reflected, and an interface or surface waves are generated that decay exponentially with depth. Only for SV waves with angles of incidence less than the critical angle, reflected primary or compression P-waves and SV waves are generated without generating interface or surface waves. Therefore, as noted in NUREG/CR-6896 (Reference 8.48), evaluations of effects of inclined seismic waves on the BWRX-300 RB SSI response and design consider the effects of SV waves with inclined angles up to the critical angle $\emptyset_{cr}$.

The effects of inclined shear waves on the SSI response and design of the BWRX-300 RB may be evaluated considering SV waves with two inclination angles equal to $\emptyset_{cr}/2$ and $\emptyset_{cr}$. The evaluation is performed in two-steps. First set of analyses are performed on a free-field model that consists of only the subgrade layers without any structures to determine the effects of inclined SV waves on the free field response including the FIRS, PBIRS, and PBSRS. If the results from the first step indicate the effects of inclined waves on the FIRS, PBIRS, and PBSRS are significant, sensitivity SSI analyses are performed with inclined SV waves on the BWRX-300 RB SSI model used for the design basis SSI analysis.

The sensitivity SSI analyses for evaluation of effects of SV are performed in frequency domain using the same BE strain-compatible subgrade profile used for the design basis SSI analysis that is developed as described in Section 5.2.4. PBSRS-compatible ATHs input motions may be applied at the ground surface elevation to avoid the mismatch in the soil column frequency between the non-vertically propagating waves and the FIRS compatible in-layer motions that are calculated considering vertically propagating shear and compression waves as described in Section 5.2.3. The angle of incidence of the input motion is defined at the top of the elastic half space.

Key seismic responses, described in Section 5.3.1, are calculated from the inclined waves sensitivity SSI analysis and compared with the results of the corresponding design basis SSI analysis of BE profile performed using vertically propagating seismic waves to assess the effects of the inclined SV waves on the RB seismic response. These key responses are also compared with the enveloping design basis demands to determine if the RB design envelopes the possible effects of non-vertically propagating seismic waves. Based on these comparisons, the seismic design demands are amplified to address effects on non-vertically propagating seismic waves if the responses calculated from the inclined waves sensitivity analyses significantly exceed (>10%) the design basis ISRS or the seismic load demands.

5.3.4 Approaches for Meeting DC/COL ISG-017 Guidance

In accordance with DC/COL ISG-017 (Reference 8.6), Section 5.2.3 guidance, the input ground motion is applied to the RB SSI model at foundation bottom elevation. A horizontal and a vertical FIRS define the magnitude and the frequency content of the input design ground motion at the RB foundation bottom elevation. As described in Section 5.2.2, the horizontal FIRS is developed based on results of probabilistic site response analyses of randomized subgrade profiles, and the

vertical FIRS is developed based on integration of horizontal hazard curves with applicable V/H spectral ratios. The horizontal FIRS is amplified if needed to ensure it meets the minimum earthquake requirement of 10 CFR 50, Appendix S, described in Section 5.2.2.

The intent of DC/COL ISG-017 (Reference 8.6) is to ensure that the deterministic SSI analysis of embedded RB structure use ground motion inputs that are hazard consistent with the results of probabilistic SRA described in Section 5.2 at the foundation bottom elevation and at ground surface. For the deeply embedded BWRX-300 RB structure, the same criterion is applied to other intermediate elevations throughout the height of the embedment to provide consistency between deterministic SSI analysis and probabilistic SRA through the whole depth of the embedment. The consistency between free field motion for the deterministic SSI analysis and probabilistic SRA is checked at the ground surface and at intermediate elevations along the embedment depth using the PBSRS and PBIRS developed as described in Section 5.2.2. The intermediate elevations are selected based on the features of the soil and rock base-case profiles used for the probabilistic SRA. The elevations corresponding to significant V_S contrasts in the SSI soil profiles are included as intermediate elevations.

Either one of the following three approaches are used for meeting DC/COL ISG-017 by:

1. performing NEI checks as described in Section 5.3.4.1, to ensure the horizontal and vertical FIRS applied to the model at the bottom of RB foundation is adequate at the ground surface and throughout the embedment depth;
2. enveloping the results of three or more sets of SSI analysis as described in Section 5.3.4.2, performed with FIRS, PBSRS and PBIRS defined input ground motions applied at the foundation bottom, ground surface and intermediate elevations, respectively; and
3. performing NEI checks as described in Section 6.3.2.1 only for the horizontal direction and using vertical free-field input motion for the SSI analysis that are constrained along the embedment depth of the soil columns based on the V/H ratios used for the probabilistic SRA and following the methodology in EPRI 3002011804 (Reference 8.44) described in Section 5.3.4.3.

Alternatively, a probabilistic SSI analysis approach may be used following the requirements of ASCE\SEI 4-16 (Reference 8.7), Section 5.5 that by definition, would satisfy the DC/COL ISG-017 guidance to ensure the SSI analysis use ground motion inputs that are hazard-consistent with the results of probabilistic SRA.

5.3.4.1 NEI Checks of FIRS Defined Input Ground Motion

The input motions applied at the bottom of the foundation are checked following the procedure described in Section 3.2.3 of the NEI white paper (Reference 8.50) to ensure the input motions for the analyses of the embedded RB structure is adequate. For the deeply embedded RB structure, these checks are carried out by comparing the deterministically propagated motions through SSI input profiles against the results of the probabilistic SRA at the ground surface and intermediate elevations throughout the embedment depth.

These checks are performed on both the horizontal and vertical components of the ground motion by performing 1-D linear elastic site response analyses on the same set of layered subgrade profiles of strain-compatible V_S and V_P as those used for the deterministic SSI analysis. The design ground

motion defined by the horizontal and vertical FIRS is propagated vertically through the V_S and V_P SSI profiles, respectively, to calculate 5% damped ARS results for the free-field response at the SSI profiles top elevation and selected intermediate elevations.

An enveloping ARS for the free-field ground motion responses at the ground surface elevation and selected intermediate elevations in the horizontal direction are calculated as envelope of results from the site response analyses of all V_S profiles used for the SSI analysis. Similarly, the results from the site response analyses of all V_P SSI profiles are enveloped to calculate an enveloping ARS in the vertical direction at ground surface and selected intermediate elevations. The horizontal and vertical enveloping ARS at ground surface are compared with the horizontal and vertical PBSRS, which are developed as described in Section 5.2.2 to represent the design ground motion at the ground surface elevation. Similarly, the horizontal and vertical enveloping ARS at selected intermediate levels are compared with the horizontal and vertical PBIRS.

If the enveloping ARS at any of the considered elevations through the RB embedment do not bound the corresponding PBSRS or PBIRS at any range of frequencies, the corresponding FIRS is increased. The checks are repeated until both the horizontal and the vertical enveloping ARS bound the corresponding PBSRS and PBIRS at all frequencies.

The NEI check requirement for the enveloping ARS to envelop the PBSRS and PBIRS, can also be satisfied by increasing the number of subgrade profiles used for the deterministic SSI analysis.

5.3.4.2 FIRS and PBSRS Defined Input Ground Motions

Another approach for satisfying the guidance of DC/COL-ISG-017 is to use the envelope of the results from multiple sets of SSI analyses of the BE, LB and UB subgrade profiles with input free-field ground motion compatible to:

a) the FIRS and applied to the SSI model at the RB foundation bottom elevation;

b) the PBSRS and applied to the SSI model at the ground surface elevation; and

c) the PBIRS calculated at selected intermediate elevations and applied to the SSI model at the corresponding elevation.

While this approach does not necessarily satisfy the consistency of the free-field motion between the deterministic SSI and probabilistic SRA analyses, it ensures that the free-field probabilistic SRA results are enveloped by the design.

5.3.4.3 V/H Based Vertical SSI Input Motion

The NEI check described in Section 5.3.4.1 is carried out in the vertical direction by propagation of P-waves. This is contrary to the observation that the vertical motions, especially for frequencies below 10Hz, are mostly attributed to vertical and inclined propagation of S-waves through the soil and rock medium and are better characterized by empirically obtained V/H ratios that are used in the development of vertical PBSRS and PBIRS. As discussed in EPRI 3002011804 (Reference 8.44), this inconsistency could lead to overly conservative adjustments to the vertical FIRS.

The approach described in EPRI 3002011804 (Reference 8.44) can be implemented to eliminate the use of overly conservative vertical motions that may arise from application of the NEI checks

to the vertical motion. The NEI check is performed only for the horizontal direction using the methodology described in Section 6.3.2.1 to ensure that the horizontal ground motion applied in the deterministic SSI analysis at the FIRS elevation is consistent with the results of the probabilistic SRA.

Deterministic SSI analyses with input control motion in the vertical direction is carried out following the methodology described in EPRI 3002011804 (Reference 8.44) and by using software that allows V/H ratio-based constraints to the vertical free field motion. V/H ratios along the embedment depth of the structure are specified consistent with those used for the probabilistic SRA to ensure that the vertical ground motion for the deterministic SSI analysis is consistent with the results of the probabilistic SRA. The vertical free-field motion in each soil and rock layer is calculated such that it is loosely constrained to the product of the V/H ratio to the corresponding horizontal motion at that layer.

The response at the free-field interaction nodes are used to check the accuracy of the vertical motion applied to the SSI model through the RB embedment depth. The ground motion ATHs obtained from the free-field nodes represent the in-column motion at the top of each embedment soil layer. Linear 1-D wave propagation analyses are performed to transform the in-column ATHs to outcrop motion ATHs. For each embedment soil layer, V/H ratios are calculated by dividing the 5% damped ARS of the vertical outcrop motion by the 5% damped ARS of the outcrop ground motion in the two horizontal directions. The resulting V/H ratios are compared with the V/H ratios that are used in the development of vertical PBSRS and PBIRS.

5.3.5 Effects of Variation of Structural Stiffness and Damping Properties

The modelling of appropriate stiffness and damping properties of the structural members in the SSI model is essential for the accuracy of the calculated seismic responses and seismic demands. The stiffness of concrete made structural members, such as reinforced concrete or SC composite members, depends on the degree of concrete cracking. Effects of concrete cracking on structural stiffness are considered in a conservative manner per SRP 3.7.2 guidance.

Effective stiffness properties assigned to the reinforced concrete members are in accordance with ASCE/SEI 43-05 (Reference 8.4), Section 3.4.1. The stiffness of reinforced concrete calculated per ACI-349-13 (Reference 8.24) are reduced based on criteria provided in Table 3-1 of ASCE/SEI 43-05 (Reference 8.4) to address the effects of cracking of reinforced concrete members. The following stress limits, recommended in ASCE/SEI 4-16 code (Reference 8.7), Section 3.3.2, are used to determine the cracking status of reinforced concrete members based on the nominal concrete compressive strength (f_c') expressed in psi and the overall level of stresses the structural member experiences under the earthquake design loads in combination with other applicable design loads:

- Wall cross section average in-plane shear cracking stress limit of 3 $\sqrt{f_c'}$ (psi), and
- Flexural cracking stress limit 7.5 $\sqrt{f_c'}$ (psi).

The effective stiffness of SC walls is determined based on AISC N690 (Reference 8.25), Section N9.2.2. The effective in-plane shear stiffness of SC walls is determined from the equations provided in AISC N690 (Reference 8.25), Section N9.2.2(b). The equation used to determine the in-plane shear stiffness is selected as follows by comparing the required membrane in-plane shear

strength per unit width (S_{rxy}) to the in-plane shear force per unit width at concrete cracking threshold (S_{cr}):

- a full (uncracked concrete) in-plane shear stiffness calculated per AISC N690 code Equation A-N9-9 is used if $S_{rxy} \leq S_{cr}$;
- an in-plane shear stiffness transitioning between uncracked and fully cracked calculated per AISC N690 code Equation A-N9-11is used if $S_{cr} < S_{rxy} \leq 2\ S_{cr}$; and
- a fully cracked in-plane shear stiffness calculated per AISC N690 code Equation A-N9-14 is used if $S_{rxy} > 2\ S_{cr}$

An effective in-plane shear stiffness equal determined from AISC N690 code Equation A-N9-12, may be used if seismic load is considered in combination with accident thermal loading.

AISC N690 (Reference 8.25), Equation A-N9-8 is used to calculate the effective flexural stiffness of SC members based on the cracked transformed section, which accounts for stiffness from the steel faceplates as well as the cracked concrete infill. This equation is also used to account for reduction of flexural stiffness due to additional concrete cracking due to conditions related to accident thermal loading. The additional reduction in flexural stiffness due to accident thermal can be ignored for operating thermal conditions where thermal gradients are small and develop over longer periods of time.

The stiffness of FEs modeling reinforced concrete or SC members are modified using the effective stiffness factor for their dominant response parameter. The members with reduced (cracked concrete) are assigned higher SSE damping values reflecting the higher dissipation of energy in these members that experience high stress responses under design earthquake loads.

In accordance with SRP 3.7.2 guidance and the requirements of ASCE 4 (Reference 8.7), Section 3.3.2, the analyses are performed on models that represent best estimate stiffness properties of the concrete made structures. Depending on the level of stress in the concrete due to the most critical seismic load combinations, effective stiffness is assigned to the concrete members depending on their cracking status.

In accordance with SRP 3.7.2 Subsection II.3.C.iv guidance, best estimate structural stiffness properties are determined as follows:

1) Reduced stiffness properties are assigned to all concrete made members that experience stresses that correspond to fully cracking conditions under dead loads and normal operational conditions alone;
2) Using BE subgrade properties, SSI analyses are performed on the model with uncracked concrete properties and lower OBE damping values that are determined to remain uncracked in Step 1 to determine their cracking status through comparison of the calculated stresses with cracking stress limits; and
3) Additional SSI analyses of the partially cracked models are performed to ensure that no significant cracking will occur in other members due to stress redistributions.

The cracking status checks in Steps 2 and 3 above are performed considering the governing seismic load combination.

As recommended in ASCE\SEI 4-16 (Reference 8.4), Section C3.3.2, the design basis model with stiffness properties that yield a conservative seismic responses and design for the site-specific conditions can also be used for addressing the effects of structural stiffness variations. For example, models with upper bound stiffness properties can be used to develop the seismic design basis for HRHF site conditions because the cracking of concrete will reduce the structural stiffness resulting in shifts of the responses to lower frequencies where the energy content of the HRHF design motion is lower. The effects of concrete cracking may also be neglected, as recommended in ASCE\SEI 4-16 (Reference 8.4), Section C3.3.2, if the frequencies in all three directions (vertical and both horizontal) of the SSI system are lower than the natural frequency of the soil column.

To further address the effects of structural stiffness variations, sensitivity SSI analyses are performed on models representing the following two bounding structural stiffness conditions:

- a fully cracked condition when all concrete structural members are fully cracked and are assigned higher SSE damping properties; and
- a fully uncracked condition when all concrete structural members are assigned full (uncracked concrete) stiffness and lower OBE damping properties.

These sensitivity analyses are performed for BE subgrade profile to evaluate possible amplifications of in-structure responses and load demands on the steel members due to the load redistribution effects. These evaluations are based on comparisons of results from these two sensitivity analyses and the design basis analysis performed for the BE profile using BE dynamic properties for the RB structure. The comparisons are performed for in-structural responses and stress demands at key locations selected, as described in Section 5.3.1.

In general, the damping ratio assigned to structural members should be consistent with their cracked or uncracked state. However, in accordance with RG 1.61, Section C.1.2, the seismic demands for the design of RB structural members can be obtained from a structural model that has all major load resisting structural members assigned with higher SSE damping properties representative of the dissipation of energy conditions in these structures when subjected to high stresses corresponding to the code design limits. The uncracked members in the models used for calculation of ISRS and other in-structure response demands for seismic design and evaluation of SC-I equipment and components, are assigned lower OBE damping values.

5.3.6 Dynamic Modeling of Subsystems, Components, and Equipment

The dynamic properties of subsystems, components, and equipment are included in the SSI analysis model based on the decoupling criteria of SRP 3.7.2 Subsection II.3.B, depending on the ratios of the mass and first natural frequency of the subsystem, component, or equipment to those of the supporting structure. The dynamic properties of the Reactor Pressure Vessel (RPV) and its components are represented by a lumped mass stick model capable of capturing all significant modes of RPV seismic response to capture dynamic coupling effects.

Depending upon the configuration, the dynamic interaction coupling between the RB structure and the secondary systems can be due to one or more of the following: mass interaction, non-classical damping, or out-of-phase motion of supports of an equipment/piping system supported at multiple nodes. These ESI effects are more pronounced when the equipment has a dominant natural mode

of vibration and its mass exceeds about 0.5% of the contributing mass of the structure at the equipment support.

For the purposes of generating ISRS and in-structure responses that provide a more realistic representation of the seismic demands on equipment, equipment-structure interaction (ESI) effects may be considered in a more explicit manner using one of the following approaches:

- Direct method, which consists of explicit modeling of the equipment mass, stiffness, and damping characteristics as one or more additional degrees of freedom in the primary model.
- Mass-impedance ESI method, in which the mass of the equipment and dynamic stiffness in the form of impedance is used to obtain an ESI-modified ISRS at the secondary system support location. This approach uses the ratio of the secondary system impedance to that of the support location on the structure to calculate a modified acceleration response for the secondary system with a given mass. This approach is used when:
  1) the secondary system dynamic response may be reasonably approximated as a single degree-of-freedom system with a single attachment point, and
  2) the dynamic interaction of the subject secondary system is not significantly affected by presence of additional secondary systems.
- Generalized ESI method, which allows for consideration of a secondary system with multiple degrees-of-freedom, attached to the structure at multiple points and having a damping ratio that is different than the supporting structure. More details about implementation of this approach are provided in EPRI Report 3002010666 (Reference 8.23).

As described in Reference 8.52 and EPRI Report 3002009429 (Reference 8.51), the mass-impedance ESI method accounts for the equipment mass and dynamic stiffness (impedance) or inversely the dynamic flexibility (compliance) in the calculations of an ESI-modified acceleration response for the considered equipment. The impedance or compliance functions at the support point are obtained for each frequency of analysis by applying a unit harmonic load in the direction of interest and obtaining the resulting complex displacement that is used to compute the stiffness and damping associated with the support response.

The structural impedance or complex stiffness, $D_s(\omega)$, at an equipment support location is expressed as:

$$D_s(\omega) = K_s(\omega) + i\omega\, C_s\,(\omega) \tag{5-27}$$

where: $K_s(\omega)$ and $C_s\,(\omega)$, respectively, are the stiffness and damping associated with the response of the support location to a unit harmonic load with frequency of analysis ω.

Similarly, the equipment impedance (D_e) is expressed in terms of its stiffness, k_e, and its damping, c_e, as follows:

$$D_e(\omega) = K_e(\omega) + i\omega\, C_e\,(\omega) \tag{5-28}$$

where: $K_e(\omega)$ and $C_e\,(\omega)$, respectively, are the equipment stiffness and damping at frequency ω.

The equipment mass and the ratio of the $D_e(\omega)$ to $D_s(\omega)$ is used to calculate the ESI-modified ISRS in frequency domain following the formulation presented in Tseng (Reference 8.52) and Chapter 5 of EPRI Report 3002009429 (Reference 8.51). The same formulation may be expanded to also obtain ESI-modified ISRS applicable for lighter equipment or components adjacent or mounted on a primary equipment.

5.3.7 Modeling of Structure-Soil-Structure Interaction Effects

The seismic soil-structure-soil interaction (SSSI) of the RB with the adjacent RwB, CB and TB can have a significant effect on the overburden pressure loads applied on the RB below grade walls and to smaller extent on the RB seismic response. The models used for seismic SSI analyses of RB include the surrounding foundations and structures to capture these SSSI effects in the RB seismic design.

Simple models representing the BE dynamic properties of surrounding buildings and foundations are included in the RB FE model used for the seismic SSI analysis. These simple models are sufficiently refined to capture all global modes of vibration of RwB, CB and TB structures with significant (> 20%) modal mass participations in the three orthogonal directions.

Section 6.0 presents the approach for addressing the requirements related to the II/I interaction of RB with the surrounding RwB, CB and TB structures and foundations.

5.3.8 Excavation Support and Backfill Effects

The BWRX-300 design does not rely on the resistance provided by the supports that may be used to secure the stability of excavation and the lean concrete fill used to fill the gaps between the below-grade RB shaft exterior wall and the excavated soil and rock. These construction elements are for temporary use and are excluded from the models used for the static and dynamic SSI analysis because they are not designed to maintain their structural integrity through the entire operational life of the plant. The exclusion of the excavation supports and fill concrete results in conservative estimate of static and dynamic lateral pressure demands on the RB below grade walls.

The stiffness of the excavation support and the fill concrete may affect the seismic response of RB structure. SA sensitivity seismic SSI analyses may be performed using BE properties of surrounding in-situ subgrade materials on an RB FE model that includes the excavation support structure and the fill concrete to assess their effect on the BWRX-300 RB seismic response. Shell and beam elements are used represent the BE dynamic properties of the excavation support structure. Solid elements are used to represent BE, and the dynamic properties of concrete fill material. The geometry of the excavation support and the lean concrete are modelled based on the nominal dimensions obtain from excavation plan drawings. Uncertainties related to the determination of concrete fill or excavation support properties and geometry may be addressed by using models for these sensitivity analyses that provide biased estimates of excavation support and fill effects on the RB seismic response.

The technique used for the RB shaft construction and waterproofing can also affect the friction at the interfaces between the RB exterior walls and the surrounding excavation support structure or fill material. In order to address the uncertainties related to the modeling of friction at the RB shaft interfaces, the sensitivity analyses may be performed considering two bounding conditions:

a. fully bonded conditions assuming no slippage between the RB shaft and surrounding materials

b. no-friction conditions assuming no friction resistance of RB shaft exterior walls

The results of these sensitivity analysis for in-structural responses and stress demands at key locations, selected as described in Section 5.3.1, are compared with the corresponding results of the design basis SSI analyses of FE model that excludes the excavation support and the fill concrete. If the comparisons show significant exceedances (> 10%) in the RB seismic response due to the interaction with the excavation support and fill concrete, the results of these sensitivity analyses are included in the RB seismic design basis.

5.3.9 Soil Separation Effects

Depending on the subsurface conditions, and magnitude and frequency characteristics of the input ground motion, there may be short instances of time when the parts of the RB below-grade exterior shaft wall separate from the surrounding soil or rock. In accordance with SRP 3.7.2 guidance, the SSI analysis of the BWRX-300 RB addresses the uncertainties related to the inability of linear models used for the seismic design SSI analysis to explicitly represent the separation between the soil and the structure.

In lieu of performing non-linear seismic SSI analysis as described in Section 5.3.11, the importance of soil separation effects can be assessed following the guidance of ASCE 4 (Reference 8.4), Section 5.1.9(b) by comparing the seismic and static lateral pressures on the RB shaft wall. To assess the extent of soil separation, the maximum lateral earth pressure, calculated form the seismic SSI analysis of BE subgrade profile, are compared with a lower bound estimate the static pressures $p_{LB}(z)$ calculated function of the depth (z) as follows:

$$p_{LB}(z) = 0.9\left[\frac{K_{0_{LB}}(z)}{K_0(z)}p_{1g}(z)\right] \qquad \text{for } z \le z_{gw}$$

$$p_{LB}(z) = \gamma_w(z - z_{gw}) + 0.9\left[\frac{K_{0_{LB}}(z)}{K_0(z)}p_{1g}(z)\right] \qquad \text{for } z > z_{gw} \tag{5-29}$$

where: γ_w is the unit weight of water

z_{gw} is the nominal groundwater table level,

$K_{0_{LB}}(z)$ is a lower bound estimate of the lateral coefficient at rest,

$K_0(z)$ is the coefficient at rest value used as input for the design analysis,

$p_{1g}(z)$ is the static lateral pressure calculated from the 1-g static SSI analysis.

In the above equation, the static lateral pressures calculated from static design SSI analysis with 1-g loading are reduced by 10% to account for uncertainties in calculation of soil unit weights and surcharge loads.

The regions where the static lateral pressure $p_{LB}(z)$ is lower than the seismic lateral pressure calculated from the seismic SSI analysis of BE soil profile indicate potential separation at the soil-structure interfaces.

To determine if the separation at soil-structure interfaces can have significant effect on the seismic response, a sensitivity analysis is performed on a model where portions of the below-grade shaft

wall may experience separation from the subgrade soil are assumed to remain unbonded for the total duration of the earthquake. The key in-structure responses and stress demands, described in Section 5.3.1, calculated from this sensitivity analysis are compared to the corresponding results of the design basis SSI analysis performed on model with BE properties representing fully bonded conditions. If the comparisons indicate that the seismic in-structure responses and stress demands from the fully separated model provide a conservative representation of the soil separation effects, exceed the design basis developed based on results of SSI analysis of fully bonded models by more than 10%, the results of this sensitivity analysis are included in the RB seismic design basis.

Alternatively, non-linear SSI analyses may be performed as described in Section 5.3.11 using non-linear gap contact elements at the SSI interfaces.

5.3.10 Groundwater Variation Effects

The seismic design of RB is based on analysis of SSI models that reflect fully saturated conditions for all soil materials located below the nominal groundwater elevation. The potential effects of groundwater level variability on the seismic design are addressed following SRP 3.7.2 Subsection I.4.H guidelines. The effects of groundwater on the dynamic properties of subgrade materials are addressed by adjusting the values of the Poisson ratios for the softer soil materials located below the groundwater table as specified in Section 5.2.4.

The potential effects of groundwater variability are assessed by comparing the seismic responses obtained from two sensitivity analyses of:

A. fully saturated soil profile with BE soil dynamic properties representative of accidental flood groundwater level; and

B. dry soil profile with BE soil dynamic properties representative of the extreme conditions when the groundwater is located below the RB foundation bottom elevation.

The results of these two sensitivity analyses for key in-structure responses and stress demands, defined in Section 5.3.1, are compared with the results of design bases SSI analyses. If the comparisons show that the effects of groundwater variation significantly exceeds (>10%) the design basis developed based on results of analysis of profiles representative of fully saturated soil below the nominal groundwater elevation, the results of these two sensitivity analysis are included in the RB seismic design basis.

5.3.11 Non-Linear Seismic Soil-Structure Interaction Analysis

The non-linear effects can be important for the RB design for sites characterized by a high seismicity and a highly non-linear behavior of subgrade materials. Sensitivity non-linear seismic SSI analyses may be performed for these sites following the guidance provided in ASCE\SEI 4-16 (Reference 8.4), Appendix B to assess the following non-linear effects on the RB seismic response and design:

(a) Secondary non-linearity of subgrade materials including non-linearities at rock discontinuities

(b) Non-linearity at soil-structure interfaces such as separation and sliding

In general, the structural vibration induces plastic deformations of the subgrade materials and dissipation of energy in the SSI system that reduces the structural response as shown in Reference 8.27 and Reference 8.28. Nevertheless, the secondary non-linearity of subgrade

materials may amplify the magnitude of the dynamic earth pressures on the RB below-grade exterior walls. In particular, the presence of fracture zones, joints, bedding planes, discontinuities and other weak zones within the rock mass may affect the stability of individual blocks or the rock mass during an earthquake and can significantly amplify the seismic rock pressure loads.

If the earth pressure load validations described in Section 5.1.3, indicate significant non-linear effects on the static earth pressure loads, non-linear SSI analyses are performed for sites with high intensity design ground motion to assess the non-linear effects on the earth pressure loads on the RB below-grade exterior walls. To capture the non-linear behavior of the soil and rock around the RB shaft, these sensitivity SSI analyses use BE non-linear constitutive models for the subgrade materials developed following the guidelines of ASCE\SEI 4-16 (Reference 8.4), Section B.4 to be consistent with:

- soil and rock constitutive models used for the non-linear FIA described in Section 4.2, and
- BE base-case models used for the probabilistic SRA described in Section 5.2.2.

If the results of sensitivity evaluations described in Section 5.3.9 indicate that the separation at soil-structure interfaces is significant, sensitivity SSI analyses may also be performed on models with contact/interface elements capable of capturing non-linearities at the soil-structure interfaces to explicitly assess the effects of possible separation and sliding at the soil-structure interfaces. The same type of contact/interface elements can be used to model the non-linear effects at rock discontinuities. The parameters assigned to the contact/interface elements are consistent with the non-linear FIA interface models described in Section 4.3.1.

Because the focus of the sensitivity non-linear SSI analyses is on the effects of subgrade material non-linearities and non-linearities at soil structure interfaces, the structural members are assigned linear-elastic properties representative of best estimate stiffness and damping properties of the structure, determined as described in Section 5.3.5.

The energy dissipation or the damping is introduced in the non-linear SSI system through:

- hysteretic energy dissipation of the non-linear elastic-plastic constitutive models used for the soil and rock materials;
- Coulomb friction or viscous damping assigned to the interface elements;
- material viscous damping assigned to the liner elastic structural model; and
- radiation damping.

The hysteretic damping is limited to 15% and 10% for soil and rock materials, respectively.

The non-linear SSI analyses also pay attention to the unintended numerical damping introduced in the SSI response solution by the numerical integration that can be manifested either by positive energy dissipation or negative energy production damping. Viscous type of damping is assigned to the elastic structural elements to capture the dissipation of energy in the RB structure. Because the viscous damping is frequency dependent and increases in proportion to the frequency, the parameters defining the viscous damping are carefully specified to adequately model the dissipation of energy in the RB structure and avoid over-damping at higher frequencies.

Model boundaries are established to adequately simulate semi-infinite subgrade conditions and account for the radiation damping, which is due to radiation of seismic waves resulting from wave reflections and oscillations/vibrations of the structures, systems, and components. Domain Reduction Method (DRM) may be used where the model is divided in two parts:

- Domain of interests that includes non-linear models of the subgrade surrounding the RB and the SSI interfaces together with a liner elastic model of the structure;
- Free-field domain that includes linear elastic model of the subgrade located far from the RB.

Viscous damping elements at the boundary may be used to account for the radiational damping. If DRM is used, the radiation damping is accounted for by assigning viscous damping to the linear elastic elements outside the domain of interest. No viscous damping is assigned to the linear elastic elements that are adjacent to the domain of interest to prevent producing potentially significant reaction forces from large viscous damping that is placed on nodes of elements that are shared with the domain of interest.

Prior to running the non-linear SSI analysis, initial conditions are established in the model as described in Section 4.3.4 representative of operational stage site subgrade conditions under static loading. Three components of the earthquake motion are applied simultaneously to the SSI model at the boundaries of the subgrade domain following the guidelines provided in ASCE\SEI 4-16 (Reference 8.7), Section B.3. The input ground motions are applied using control motion force time histories.

To estimate the responses of the SSI system during a typical design level earthquake, a set of control motions are used for the sensitivity evaluations that are consistent with the ground motion levels considered for development of strain-compatible properties in Section 5.2.4. These motions are applied to the following two models:

(1) The non-linear SSI model to predict the non-linear response of the SSI system

(2) Linear-elastic model with configuration and properties of the model used for the design basis SSI analysis of BE equivalent-linear subgrade profile

The results of analyses of these two SSI models for key in-structure responses and stress demands, defined in Section 5.3.1, are compared to assess the significance of the subgrade non-linearities on the RB seismic response. The results of the analyses of the non-linear SSI model (1) are also compared to the corresponding 5% damped enveloping ISRS and stress demands used for the design to evaluate the effects of subgrade non-linearities on the RB seismic design. If these comparisons show that the non-linear effects significantly exceed (>10%) the seismic design, the equivalent linear models used for the design basis SSI analyses are adjusted to provide responses that envelope the non-linear effects.

Alternatively, the analyses performed to evaluate the non-linear effects may also use broad-frequency band control motions that result in free-field motion at the surface of the model compatible to the PBSRS to obtain biased estimates of the non-linear effects on the RB seismic response and design. As described in Section 5.2.2, the FIRS, PBSRS and PBIRS are broad-band spectra that define the seismic ground motion for the design of the BWRX-300 RB as an envelope of spectra of multiple design level earthquakes. Therefore, the use of input control motions that

are compatible to the PBSRS, results in overdriving the subgrade non-linear response and biased overestimate of the subgrade stiffness degradation and the dissipation of energy in the SSI system. Limitations may be imposed on the soil and rock hysteretic damping to control dissipation of energy in the SSI system.

Prior to running the non-linear SSI analyses, validation analyses are performed on a free field SSI model of the subgrade alone to demonstrate that:

- the amplitude and the frequency content of the control motions applied to the model are representative of design level earthquake event ground motions; and
- the non-linear constitutive models represent free-field responses of the subgrade that is constant with the equivalent linear subgrade properties used as input for the design basis linear-elastic SSI analysis.

Control motions applied to the free-field validation model are consistent with the ground motion levels considered for development of strain-compatible properties in Section 5.2.4. The validation analysis provides 5% damped spectra of the horizontal and vertical free-field responses at the FIRS, PBSRS and PBIRS elevations. Comparisons of these 5% damped spectra with the corresponding results of probabilistic SRA and the ground motion design spectra, described in Section 5.2.2, are made to calibrate the non-linear SSI model and ensure the inputs used for the evaluation of non-linear effects are consistent with the ground motion and site parameters used for the seismic design.

5.4 Design Analyses Summary

The following innovative approaches implemented for the BWRX-300 design analyses presented in this section of the report may be referenced during future licensing activities:

(1) Overall one-step analysis approach presented in Section 5.1 for the BWRX-300 RB deeply embedded design, including the analysis assumptions provided in Section 5.1.2 that are beyond the guidance of SRPs 3.7.2, 3.8.4 and 3.8.5.

(2) One-step FE modeling requirements provided in Section 5.1.1 that are specific for the static and seismic SSI analysis of the deeply embedded BWRX-300 RB structure, including contact springs along the embedded RB shaft that are beyond the guidance of SRPs 3.7.2, 3.8.4 and 3.8.5.

(3) Innovative deterministic approach presented in Section 5.1.3 that uses the results of the non-linear FIA, described in Section 4.3.4.5, to ensure the linear elastic design SSI analyses provides conservative earth pressure design demands on the deeply embedded BWRX-300 RB structure that are beyond the guidance of SRPs 2.5.4, 3.7.1, 3.7.2, 3.8.4 and 3.8.5.

(4) Innovative probabilistic evaluation approach presented in Section 5.1.4, for demonstrating that the margins in the design of deeply embedded BWRX-300 RB structure are adequate to address uncertainties in the earth pressure load calculations that are beyond the guidance of SRPs 3.7.1, 3.7.2, 3.8.4 and 3.8.5.

(5) Guidelines and recommendations provided in Section 5.2.1 that are used for developing equivalent linear soil and rock properties as input to the BWRX-300 static SSI analyses that are beyond the guidance of SRPs 2.5.4 and 3.8.5.

(6) Requirements and methodologies provided in Section 5.2.2 that are used for developing 5% damped spectra that define the ground motion along the depth of the deeply embedded BWRX-300 RB, including the guidelines for performing NUREG/CR-6728 (Reference 8.40), Approach 3 SRA that are beyond the guidance of SRP 3.7.1 and DC/COL-ISG-017.

(7) Requirements provided in Section 5.2.3 for developing ground motion time histories for use as input to the seismic SSI analyses, including the additional requirement of five sets of acceleration time histories to mitigate the uncertainties in the computed responses due to the phasing of the time history frequency components, and refinement of time step ensures the accuracy of the calculated high-frequency in-structure responses that are beyond the current guidance of SRP 3.7.1.

(8) Methodology provided in Section 5.2.4 for developing a suite of subgrade profiles of strain-compatible dynamic properties for use as input to the seismic SSI analyses based on the results of the NUREG/CR-6728 (Reference 8.40), Approach 3 SRA that are beyond the current guidance of SRP 3.7.1.

(9) General methodology and guidelines for seismic SSI analysis of deeply embedded SMR is presented in Section 5.3, including guidelines for selection of: (1) key structural responses for evaluation of SSI responses (Section 5.3.1); and (2) frequencies of analysis, (Section 5.3.2) that are beyond the current guidance of SRP 3.7.2.

(10) Comprehensive approach provided in Section 5.3.3 for evaluating the effects of non-vertically propagating seismic waves on the design ground motion and seismic response of the deeply embedded RB structure that are beyond the guidance of SRPs 2.5.4, 3.7.1 and 3.7.2.

(11) Three different approaches provided in Section 5.3.4 for ensuring the results from the deterministic SSI analyses of the RB structure are consistent with the results from the probabilistic SRA that are beyond the current guidance in DC/COL-ISG-017.

(12) Recommendations for SSI parameter evaluations of the effects of concrete cracking, soil-structure interface conditions, soil separation and groundwater variations on the seismic response of the deeply embedded RB structure (Sections 5.3.5, 5.3.8, 5.3.9 and 5.3.10, respectively), including the approaches for their inclusion in the design that are beyond the guidance of SRP 3.7.2.

(13) Recommendations provided in Section 5.3.6 for considering ESI effects in the development of in-structure seismic response demands for equipment design and qualification that are beyond the current guidance of RG 1.122.

(14) Modelling requirements provided in Section 5.3.7 for considering SSSI effects of deeply embedded RB structure with adjacent structures and foundations that are beyond the current guidance of SRP 3.7.2.

(15) Recommendations for performing non-linear seismic SSI analyses presented in Section 5.3.11 for evaluating the effects of soil separation and soil secondary non-linearity on seismic response and design of the deeply embedded RB structures constructed at sites characterized by a high seismicity and a highly non-linear behavior of subgrade materials that are beyond the current guidance of SRPs 3.7.1 and 3.7.2.

6.0 DESIGN APPROACH FOR II/I INTERACTION

This section presents a graded approach with accompanying acceptance criteria for the design and II/I interaction evaluations of the CB, TB and RwB structures that are adjacent to the deeply embedded SC-I RB structure. CB, TB and RwB structures that are in close proximity to the SC-I RB are designed in accordance with their seismic classification as described in Section 6.1. Additional design evaluations are performed for SSE, tornado, and hurricane extreme wind condition to ensure the CB, TB and RwB structures meet the following Seismic Category II/I interaction guidance of SRP 3.7.2 Subsection II.8:

- the CB, TB and RwB structures do not collapse or collide with the RB to impair RB structural integrity or the safety functions of RB SC-I SSCs or compromise the safety functions of those SSCs that are required to remain functional following an SSE event;
- the CB structure does not collapse to result in incapacitating injury to the control room occupants; and
- the TB structure does not collapse to result in impairment of safety functions of the main steam line.

II/I interaction evaluations are performed by evaluating the lateral load-resisting systems of the CB, RwB, and TB to ensure their ability to prevent adverse interactions when subjected to SSE, tornado wind and tornado missiles loadings specified by RG 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," Revision 1, and hurricane and hurricane missiles loadings specified in RG 1.221, "Design-Basis Hurricane and Hurricane Missiles for Nuclear Power Plants," Revision 0. These evaluations demonstrate:

1. no gross failure of the CB, RwB, and TB structures; and
2. gap distances between adjacent RB, CB, RwB, and TB structures can accommodate the resulting structural displacements.

As described in Section 6.1, CB, TB and RwB structures are designed to exhibit an elastic response for design basis loadings, which are of smaller magnitudes than those used for the design of SC-I RB structures. In accordance with SRP 3.7.2 Subsection II.8 guidance, II/I interaction evaluations are performed considering limited inelastic deformations and SC-I seismic and extreme loads as described in Sections 6.2 and 6.3, respectively.

Per SRP 3.7.2 Subsection II.8, Criterion C guidance, the gaps between the RB and adjacent structures are considered adequate, if the gap provided is larger than the absolute sum of the displacements of each structure for the analyzed loading. Furthermore, the gaps are evaluated along the entire height of the adjacent structures and are designed to consider construction tolerances. The consideration of construction tolerances is meant to ensure that as-built gaps are within the limits of the as-designed gaps. The prevention of gross failure is met by following governing design codes and standards as described in Sections 6.2 and 6.3 for seismic and extreme wind interaction checks, respectively.

6.1 Control Building, Turbine Building and Radwaste Building Design Bases

CB, TB and RwB structures and foundations are designed in accordance with their seismic classification:

- Non-Seismic Category for the CB and TB structures; and
- RW-IIa Category for the RwB structure.

6.1.1 Non-Seismic Control Building and Turbine Building Structures and Foundations Design Bases

The non-seismic CB and TB structures are designed in accordance with the IBC (Reference 8.53). IBC (Reference 8.53), Chapter 16 provides structural design requirements, including those related to structural loads and load combinations, which also rely on applicable provisions from ASCE 7-16 (Reference 8.54). IBC (Reference 8.53), Section 1901.2 states that structural concrete shall be designed in accordance with the requirements of IBC Chapter 19 and ACI 318-14 (Reference 8.55) as amended in Section 1905 of the IBC. Concrete structures include the reinforced concrete foundations of the CB and TB, TB concrete pedestal on an independent foundation, and TB concrete walls used for radiation shielding and missile protection. The control room may also consist of a reinforced concrete structure within the steel framed structure of the CB. Section 2205.1 of IBC (Reference 8.53) invokes AISC 360 (Reference 8.58) for the design, fabrication and erection of structural steel elements in buildings, structures and portions thereof. Steel structures include the CB and TB steel braced frames with roof deck diaphragms.

In accordance with IBC (Reference 8.53), Table 1604.5, both the CB and the non-seismic portion of the TB are designated Risk Category IV structures. The CB and TB are considered part of a power-generating station required as emergency backup for Risk Category IV structures. Furthermore, the control room in the CB is designated as an emergency shelter to protect the inhabitants of the control room both during and after an earthquake, tornado, or hurricane.

IBC (Reference 8.53), Section 1609, describes requirements for determining wind loads and invokes ASCE 7-16 (Reference 8.54), Chapters 26 to 30. It should be noted that ASCE 7-16, Section 26.14, states that tornadoes have not been considered in its wind load provisions. Commentary in ASCE 7-16, Section C26.14, provides a discussion of tornado wind loads for building owners that may desire providing a greater level of occupant protection or minimizing building damage caused by tornadoes. Commentary in ASCE 7-16, Section C26.14.2, discusses the differences in wind pressures induced by tornadoes versus other windstorms.

The IBC (Reference 8.53) does not require the consideration of tornado wind loads except for structures designated as storm shelters, whose requirements are described in IBC (Reference 8.53), Section 423. IBC Section 1604.10, states that loads and load combinations on storm shelters shall be determined in accordance with ICC 500 (Reference 8.56). ICC 500 (Reference 8.56), Figure 304.2(1), provides tornado shelter design wind speeds, which range from 130 mph to 250 mph. ICC 500, Figure 304.2(2) provides hurricane shelter design wind speeds, which range from 160 mph to 235 mph. Of particular importance when designing for tornado wind loading is the effect of atmospheric pressure change (APC), which is described in ICC 500, Section 304.7.

IBC, Section 1613, describes requirements for determining earthquake loads and invokes ASCE 7-16 (Reference 8.54), Chapters 11, 12, 13, 15, 17 and 18, as applicable. The seismic

design category of a structure may be determined according with IBC, Section 1613, or ASCE 7-16.

6.1.2 Radwaste Category IIa Building Structure and Foundations Design Basis

The RwB is classified as a RW-IIa structure because it contains SSCs used for managing and containment of highly radioactive gas, liquid, and solid materials whose failure, considering the maximum inventory, would result in a potential unmitigated radiological release levels that may be higher than those specified in RG 1.143, Section 5.1.

In accordance with RG 1.143, Table 1 guidance, the design of the BWRX-300 RwB steel structures follows the provisions of AISC N690 (Reference 8.25). The design of the RwB concrete structures and basemat is in accordance with ACI 349-13 (Reference 8.24). Based on RG 1.143, Table 2, the loads for the design RW-IIa RwB structure includes:

- one-half of the SSE seismic load;
- wind load according to ASCE 7-16 (Reference 8.54)* for a Risk Category III structure;

  (*Note: ASCE 7-95 is referenced in RG 1.143, but most recently, ASCE 7-16 is used)
- tornado wind load equal to three-fifths of load provided in RG 1.76, Table 1; and
- Schedule 40 pipe and automobile tornado missiles based on SRP 3.5.1.4, "Missiles Generated by Tornadoes and Extreme Winds," Revision 4.

RG 1.143, Table 2, does not specify specific hurricane wind loading beyond that found using ASCE 7-16 (Reference 8.54) for a Risk Category III structure. RG 1.143, Table 3, provides the design load combinations based on the safety class of RW-IIa. RG 1.143, Table 4, provides guidance for calculating the design capacity based on safety class and the governing code/standard.

6.2 II/I Seismic Interaction Evaluations

The II/I seismic interaction evaluations of CB, TB and RwB structures are performed to ensure:

- the integrity of structural members of CB, TB and RwB lateral load resisting system under SSE loading is not compromised;
- the stability of CB, TB and RwB foundations under SSE loading is not compromised; and
- the gap distances between the CB, TB and RwB with the RB are adequate to prevent physical interactions between the buildings.

The II/I seismic interaction evaluations are based on seismic responses of CB, TB and RwB obtained from SASSI analyses of linear elastic FE models, which are refined sufficiently to provide accurate stress demands on the major lateral load resisting structural members and accurate seismic displacements in the direction of the adjacent RB. These SASSI analyses are performed on surface mounted models that neglect the effect of basemat embedment using PBSRS defined ground motions time histories and strain-compatible soil properties that are developed as described in Sections 5.2.3 and 5.2.4, respectively.

In lieu of SSI analyses, fixed base analyses can be performed, if the site-specific conditions meet any of the following criteria in ASCE\SEI 4-16 (Reference 8.7), Section 5.1.1:

1. The dominant fixed-base frequency of CB, TB or RwB structure is less than half the dominant frequency of the site-specific SSI dynamic system, assuming an equivalent rigid structure with the same mass that is supported on soil springs based on ASCE\SEI 4-16, Table 5-2;
2. For rock sites with $V_S \geq 3{,}500\ \mathrm{ft/sec}$ and where the combination of earthquake input motion, rock conditions, and structure characteristics is demonstrated to behave as a fixed-base system; or
3. For rock sites whose $V_S \geq 8{,}000\ \mathrm{ft/sec}$ at a shear strain of 10^{-4} % or smaller regardless of the frequency content of the free-field motion.

SSE demands for II/I interaction evaluations of CB, TB and RwB structures may be obtained from FE models with higher SSE damping and lower (cracked concrete) stiffness properties corresponding to limited inelastic stress responses. Alternatively, the results of seismic analyses performed for RwB and TB to develop seismic demands for design and qualification of RW-IIa SSCs may be used to develop demands for II/I interaction evaluations of RwB and TB structures. The results of these analyses, which are performed on models with lower OBE damping and full (uncracked concrete) stiffness using as input one half SSE ground motion, are multiplied by two to conservatively calculate SSE demands for the II/I seismic interaction evaluations.

If the SSSI effects on the TB, RwB or CB seismic response are significant, results of SSSI analyses described in Section 5.3.6 can also be used for the calculations of seismic demands on CB, TB and RwB structural members.

The II/I seismic interaction evaluations of CB, TB and RwB structures to resist SSE loads are based on demands obtained from the results of seismic response analyses considering limited inelastic responses. Seismic demands are used for the II/I seismic interaction evaluations of the lateral structural members of the CB, RwB and TB lateral load-resisting systems that correspond to Limit State C (LS-C) responses, which are defined in ASCE/SEI 43-05 (Reference 8.4), Table 1-4 as responses associated with limited permanent deformations and minimal damage.

To account for the inelastic response, the SSE demands obtained from the results of linear elastic seismic response analyses of CB, TB and RwB structures are reduced, based on structural system, using LS-C inelastic energy absorption factors provided in ASCE/SEI 43-05 (Reference 8.4), Table 5-1. The reduced SSE demands are combined with non-seismic demands to evaluate the structural integrity of the CB, TB and RwB lateral load resisting systems per governing nuclear design codes, ACI 349-13 (Reference 8.24) and AISC N-690 (Reference 8.25) for the reinforced concrete and steel structures, respectively.

Sliding and overturning stability evaluations are performed using the results of the seismic analyses to demonstrate the seismic stability of CB, TB and RwB foundations and ensure no physical interaction between these buildings and the RB under SSE conditions. No reductions are applied to seismic driving force demands used for the stability evaluations to account for inelastic responses of CB, TB and RwB structures.

The gaps between the RB and adjacent structures are evaluated per SRP 3.7.2 Subsection II.8, Criterion C guidance to ensure no physical interaction between the SC-I RB structure and surrounding non-SC-I structures. The gap distances are considered adequate if they are larger than the absolute sum of the SSE driven displacement of each structure in the direction toward one

another. The gaps are evaluated along the entire height of the adjacent structures considering construction tolerances. The consideration of construction tolerances is meant to ensure that as-built gaps are within the limits of the as-designed gaps. Therefore, to preclude physical interaction, the maximum allowable displacement of CB, TB, or RwB structures (Δ_{allow_i}) in the direction of the adjacent RB at floor elevation i can be formulated as follows:

$$\Delta_{allow_i} = (\Delta_{gap} - \Delta_{tol}) - \Delta_{RB_i} \quad (6\text{-}1)$$

where: Δ_{gap} is the minimum specified gap between the BWRX-300 CB, TB, or RwB and the adjacent RB

Δ_{tol} is the construction tolerance considered for the specified gap between the BWRX-300 CB, TB, or RwB and the adjacent RB

Δ_{RB_i} is the maximum RB horizontal seismic displacement at floor elevation i relative to the free field motion in the direction of the adjacent CB, TB or RwB that is obtained from the enveloping results of the RB SSI analyses described in Section 5.3

For each major floor elevation i, the maximum allowable displacement Δ_{allow_i} is compared with the maximum displacements (Δ_{II_i}) of non-SC-I category structures, surrounding the RB (CB, TB and RwB) that are estimated as follows:

$$\Delta_{II_i} = C_{NL}\,\Delta_{St_i} + \Delta_{Fe} + \Delta_{Fs_i} \quad (6\text{-}2)$$

where: C_{NL} is a coefficient that relates the seismic displacements calculated from seismic analysis of linear elastic structural models to the corresponding inelastic displacements

Δ_{St_i} is the maximum horizontal seismic displacement of non-SC-I category structure at floor elevation i, relative to the center of its basemat in the direction of the adjacent RB that is obtained from the enveloping results from the seismic response analyses of the CB, TB or RwB linear elastic model

Δ_{Fe} is the maximum horizontal seismic displacement of non-SC-I category foundation relative to the free field motion in the direction of the adjacent RB that is obtained from the CB, TB or RwB SSI analyses

Δ_{Fs_i} is the maximum horizontal displacement of the non-SC-I category structure at floor elevation i, in the direction of the adjacent RB due to possible differential settlement, sliding or rocking of its foundation

A conservative value of 1.8 may be adopted for the coefficient C_{NL} based on the maximum value specified in ASCE 41-17 (Reference 8.57), Table 7-3 for the modification factors C_1C_2 that relate maximum inelastic displacements to linear elastic displacements accounting for effects of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on maximum displacement. Displacements Δ_{Fs_i} that are due to potential sliding and/or rocking of the respective foundations are obtained from the results of the seismic stability analyses of CB, TB, and RwB foundations.

The gap distances between the RB and the adjacent CB, TB and RwB are determined to be adequate to prevent physical interactions between the buildings if the maximum displacements $\Delta_{allow_i} < \Delta_{II_i}$ of CB, TB and RwB structures at all floor elevations i.

6.3 II/I Interaction Evaluations for Extreme Wind Loads

II/I interaction evaluations of CB, RwB, or TB structures are performed to ensure no gross failure under the controlling extreme wind loading, which otherwise could impair the structural integrity or safety functions of the adjacent SC-I SSCs. Except for the selection of the wind loading, the II/I interaction checks for extreme wind loading are performed in accordance with each structure's governing design codes and standards. II/I interaction evaluations for extreme wind loading may be performed using the same analysis models as those used for the seismic II/I interaction evaluations, except soil-structure interaction effects are not needed, and appropriate boundary conditions are placed at the structure to foundation interface.

The hurricane and tornado wind loads used for the design of SC-I structures are evaluated to determine the controlling extreme wind loading. RG 1.76, Figure 1 and Table 1 divides the contiguous United States into three separate regions: Region I, Region II, and Region III. A maximum tornado windspeed of 230 mph for SC-I structures is based on the provisions on Figure 1 and Table 1. The maximum design-basis hurricane windspeeds for SC-I structures are based upon the provisions of RG 1.221, Figures 1, 2, and 3, which provide nominal 3-second gust windspeeds at 33 feet above ground over open terrain. The maximum windspeeds vary from 220 to 290 mph along the United States coastline starting from the Texas Gulf to the Florida Atlantic ending at Cape Hatteras North Carolina; and from 220 mph to 170 mph from Cape Hatteras to the northeastern tip of Maine. The annual exceedance probability of tornado and hurricane windspeeds is 10^{-7} per RG 1.76 and RG 1.221 respectively.

The maximum design-basis windspeed for non-SC-I structures is 200 mph based on ASCE 7-16 (Reference 8.54), Figure 26.5-1D for Risk Category IV structures, while the annual exceedance probability is 3.33×10^{-4}. These wind speed values are nominal 3-second gust wind speeds at 33 feet above ground for Exposure Category C, which corresponds to open terrain as described in ASCE 7-16 (Reference 8.54), Section 26.7. The controlling extreme wind loading is the loading that results in the largest deformations of the structure, while members of the lateral force-resisting system remain within applicable code limits. Missile impact effects are assessed for local damage and structural response for SC-I structures based on the design-basis tornado missile spectrum defined in RG 1.76 but are not an II/I interaction design evaluation consideration.

The main wind force resisting systems of the CB and TB are steel braced frames designed in accordance with AISC 360-16 (Reference 8.58) for design-basis loadings from ASCE 7-16 (Reference 8.54). For the interaction checks for controlling extreme wind loading, limited inelastic response of the steel braced frames is permitted as long as global stability is also confirmed. The inelastic response of the steel braced frames of the CB and TB are determined in accordance with the requirements of AISC 360-16 (Reference 8.58), Appendix 1.3. AISC 360-16, Appendix 1.3 provides general, ductility, and analysis requirements for performing steel structure's design by inelastic analysis for tornado and hurricane wind loading defined above. In addition to preventing interaction with the RB due to extreme wind loading, this approach also ensures that the CB structure does not collapse and cause incapacitating injury to the control room

occupants, and the TB structure does not collapse and impair safety functions of the main steam line.

As described in Section 6.1.2, the design-basis tornado wind load for the RwB is equal to three-fifths of load provided in RG 1.76, Table 1, and the design basis standard is ACI 349-13 (Reference 8.24). ACI 349.3R (Reference 8.18) requires structures to remain elastic under analyzed loading, so there are no provisions to consider inelastic responses for non-seismic loading; therefore, the extreme wind II/I checks for the RwB are performed to determine the maximum deflections of the RwB due to the controlling extreme wind loading, while requiring the RwB to maintain a linear elastic response to the loading.

The SC-I RB is also designed for design-basis hurricane- and tornado-generated missiles, as applicable. The hurricane-generated missiles are based on the missile spectrum in RG 1.221, Table 1 and the corresponding missile velocities in RG 1.221, Table 2. The tornado-generated missiles are based on the missile spectrum and missile velocities in RG 1.76, Table 2. The missile spectrum used for hurricane- and tornado-generated missiles is the same, except the hurricane missile spectrum only considers the larger automobile missile and not the smaller automobile missile used for Region III tornadoes. Extreme wind missiles are not considered in II/I interaction checks for the following reasons:

- II/I interaction evaluations for extreme wind ensure no gross failure of CB, RwB, or TB structures, while missile loads would only result in localized effects;
- The missile spectrum considered in the design of RB would envelope the effects of any missiles generated by localized failure of CB or TB components/cladding;
- The RwB is designed for design-basis tornado-generated missiles; and
- Any safety-related SSCs housed in non-SC-I structures have adequately designed missile barriers.

6.4 Summary of Design Approach for II/I Interaction

The following aspects of the BWRX-300 graded design approach for II/I interaction of non-SC-I CB, TB and RwB with adjacent SC-I RB presented in this section of the report may be referenced during future licensing activities that are beyond the current guidelines in RG 1.29, "Seismic Design Classification," Revision 5:

(1) General criteria for design of Non-Seismic CB and TB structures provided in Section 6.1.1, including the requirements for determining seismic and wind design loads.

(2) General criteria for design of the RW-IIa RwB structure provided in Section 6.1.2, including the requirements for determining seismic, wind, tornado wind and missile design loads.

(3) Approach for seismic II/I interaction evaluations of CB, TB and RwB structures presented in Section 6.2, including criteria and recommendations for calculations of seismic stress demands and displacements.

(4) Approach for II/I interaction evaluations CB, TB and RwB structures for extreme wind loads, presented in Section 6.3, including criteria and recommendations for consideration of wind loads.

7.0 BWRX-300 GENERIC DESIGN APPROACH

This section presents the methodology for development of generic seismological and geotechnical site parameters representing a wide range of types and conditions existing at candidate sites across North America. Section 7.1 describes the overall approach implemented for the conceptual design of BWRX-300 structures that ensures a cost-effective design applicable for a wide range of site conditions.

Generic Design Response Spectra (GDRS) and generic subgrade dynamic properties used for the conceptual design seismic analyses of BWRX-300 RB are provided in Sections 7.2 and 7.3, respectively. Section 7.4 provides generic static properties for different subgrade materials considered for the conceptual design of the BWRX-300. These generic static properties are correlated to the generic dynamic subgrade profiles (described in Section 7.5) to develop generic profiles of static subgrade properties for use as input for the conceptual design static SSI analyses. Sections 7.6 and 7.7 present the friction coefficient values and groundwater table elevations for the generic conceptual design evaluations of BWRX-300.

7.1 BWRX-300 Structural Conceptual Design Approach

Studies have shown that material quantities required for the construction of a typical Light Water Reactor (LWR) plant are a significant driver for both direct cost and schedule duration. Utilizing an innovative approach focused on cost, the BWRX-300 is conceptually designed to meet an economically viable cost target by reducing the required material quantities. Conceptual design evaluations of the BWRX-300 are being performed to evaluate the applicability of the innovative design solution for a wide range of seismological and geotechnical site conditions representing a majority (> 80%) of North American candidate sites.

An innovative conceptual design solution has been developed for the BWRX-300 RB structure that significantly reduces the building volume, concrete, and steel requirements typical for existing LWR plants:

- The majority of BWRX-300 safety important equipment and components are hosted in a below ground shaft to mitigate the effects of possible external impacts, such as aircraft, adverse weather, or earthquake.
- Sizes of structural members are limited to material quantities determined considering a cost target that leads to construction of an economically viable BWRX-300.

A FE model of an embedded BWRX-300 RB structure is developed based on the conceptual design solution.

A set of generic seismological and geotechnical site parameters are developed for use as input for the generic conceptual design and construction cost evaluations that ensure the design of the BWRX-300 is cost effective and adequate for a wide range of types and conditions existing at candidate sites across North America. Three sets of Generic Design Response Spectra (GDRS), presented in Section 7.2, define the horizontal and vertical components of the seismic design ground motion at sites with firm, medium, and hard subgrade stiffness properties. Eight generic profiles of dynamic and static subgrade properties, presented in Sections 7.3 and 7.5, respectively, provide a realistic representation of the various geotechnical conditions existing at the candidate sites for construction of the BWRX-300.

Static and seismic analyses are performed on the RB FE model following the one-step approach described in Section 5.1, for the generic profiles of static and dynamic soil and rock properties. Seismic responses obtained from eleven sets of generic seismic design SSI analyses, listed in Table 7-1, ensure the BWRX-300 seismic design is adequate for a majority of candidate sites. A set of static 1-g analyses are performed for the eight different generic profiles of static soil and rock properties presented in Sections 7.4 and 7.5.

Static and seismic load demands obtained from these generic conceptual design analyses are used to evaluate the applicability of the RB design for site conditions determined by the site location and geology. The applicability of the generic conceptual design for the variety of considered generic site conditions ensures the design of BWRX-300 is economically viable for majority of North American candidate sites.

Table 7-1: Matrix of Generic Seismic Design Site Conditions

SSI Analysis Case No.	Subgrade Profile					Seismic Region [4]	GDRS
	Name	Nominal $\bar{V}s_{30}$ [1] (m/sec)	Top Soil				
			Depth (m)	Vs_{ave} [2] (m/sec)	f_S [3] (Hz)		
1	180-600	180	610	529	0.22	WUS	Firm
2	270-60	270	61.0	340	1.4	WUS	
3	760-15	760	15.2	629	10.3	WUS	
4	400-300	400	305	731	0.60	CEUS	Median
5	500-21	500	21.3	328	3.9	WUS	
6	760-15	760	15.2	629	10.3	WUS	
7	900-8	900	7.6	676	22.2	WUS	
8	500-21	500	21.3	328	3.9	CEUS	Hard
9	760-60	760	60.6	912	3.8	CEUS	
10	900-8	900	7.6	676	22.2	WUS	
11	2032-30	2032	30.5	2051	16.8	CEUS	

NOTES:

(1) $\bar{V}s_{30}$ - measured small-strain shear wave velocity of the top 30 meters of soil

(2) Calculated from Equation (7-2)

(3) Calculated from Equation (7-3)

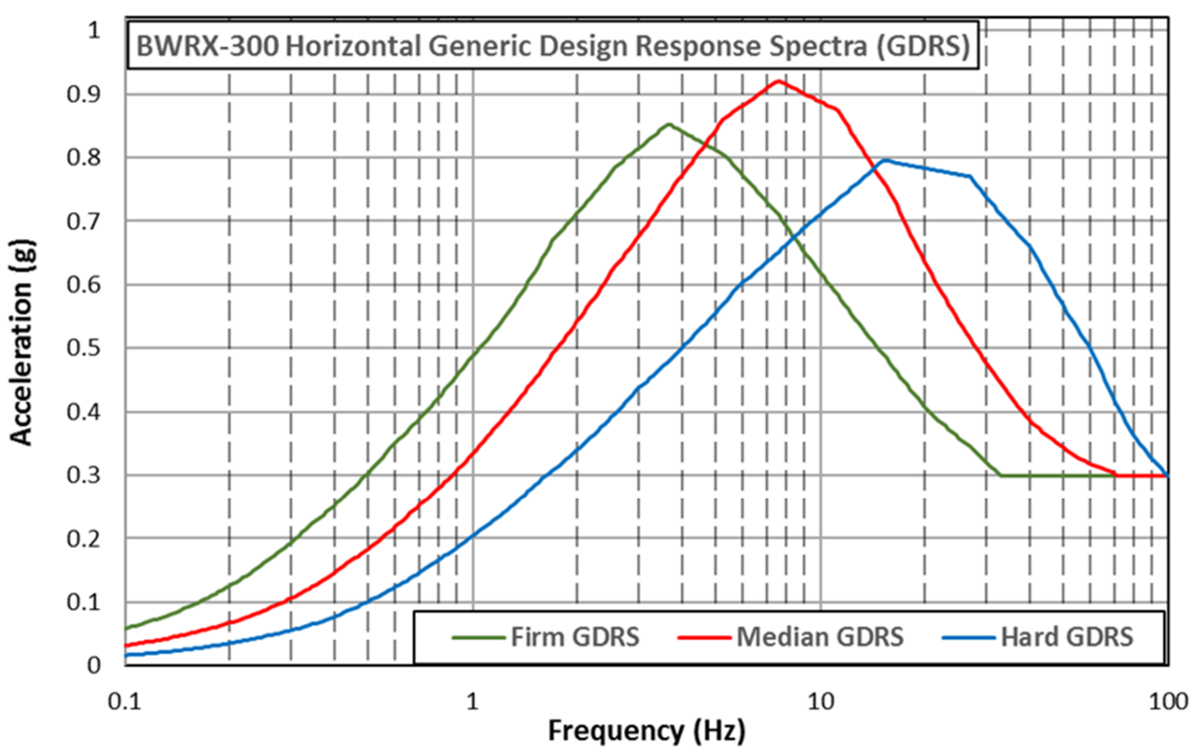
(4) WUS – Western United States, CEUS - Central and Eastern United States

7.2 BWRX-300 Generic Design Response Spectra

The ground motion for the generic design of BWRX-300 is defined by three sets of GDRS that accommodate a wide range of sites with low-frequency amplification (deep soft profiles) and high-frequency amplification (shallow soft and stiff profiles) as well as small and large magnitude contributing sources. The multiple GDRS also accommodate the differences in spectral shape between Western United States (WUS) and Central and Eastern United States (CEUS). Figure 7-1 shows the GDRS defining the horizontal and vertical components of the ground motion at the ground surface elevation for the generic BWRX-300 design located at firm, median, and hard sites.

Using three sets of GDRS ensures both a wide applicability of the generic design and helps eliminate excessive conservatism in the generic SSCs design that otherwise would be introduced using a single broadband GDRS. The results of the study presented in SMiRT-22 "Generic Input for Standard Seismic Design of Nuclear Power Plants," (Reference 8.59) showed that the generic design SSI analyses with multiple GDRS provide a more realistic representation of the seismic response at different sites and thus help achieve the goal of reaching a cost effective design of BWRX-300 SC-I SSCs.

The horizontal GDRS are anchored at 0.3g PGA. As noted in SMiRT-22 (Reference 8.59), the PGA value was adopted based on a review of publicly available data for ground motions at existing nuclear power plants and other nuclear facilities. The PGA value of 0.3g appropriately reflects an upper range in seismic hazards for CEUS sites and is representative of the overall average hazard for WUS sites. It may not envelope high seismic hazard sites, such as sites in California or other sites near active large earthquake sources.



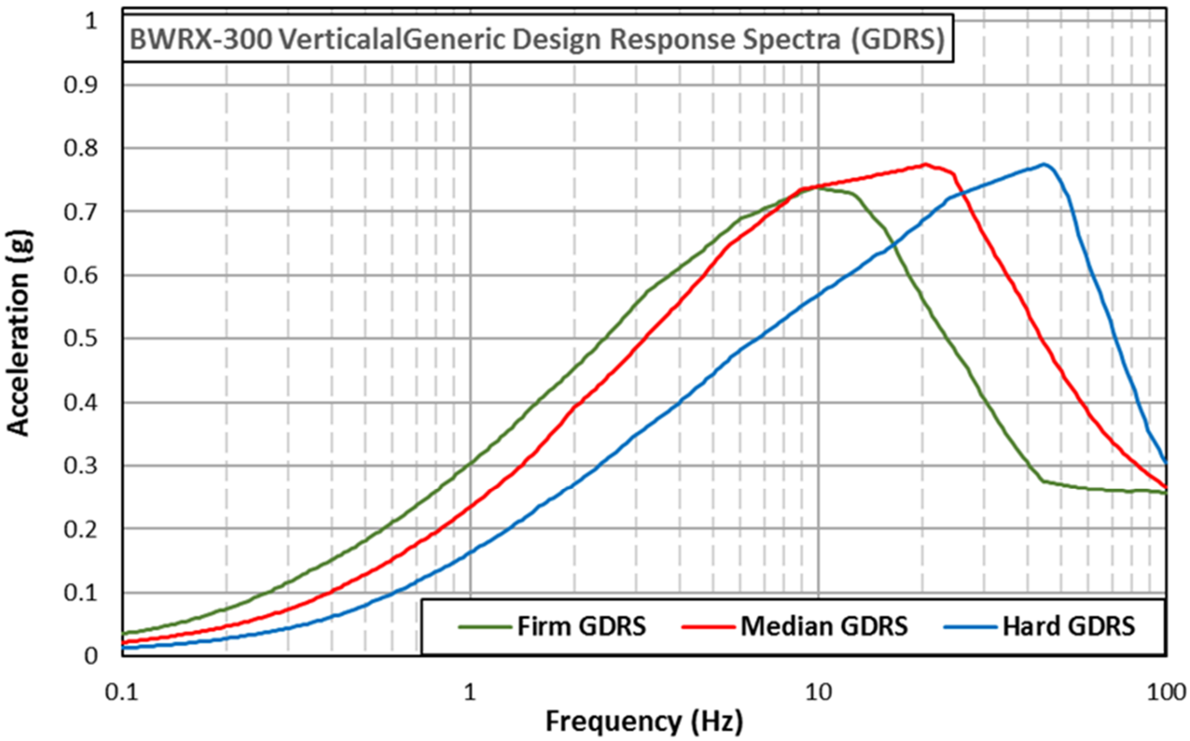


Figure 7-1: BWRX-300 Generic Design Response Spectra

The spectral shapes of the three GDRS are developed as multiple envelopes of median spectra and are computed using median site amplification functions obtained from equivalent-linear site response analyses described in SMiRT-22 (Reference 8.59). The probabilistic site response analyses are performed on suites of randomized generic subgrade profiles of measured subgrade properties. These properties are developed, as described in Section 7.3, to be representative of different subgrade conditions at firm, median, and hard sites.

Control motions for the site response analyses consisted of soft rock spectra for the analyses of WUS sites and hard rock spectra for the analyses of CEUS sites as illustrated in NUREG/CR-6728 (Reference 8.40). An earthquake magnitude M 6.5 was considered with spectral shape reflective of a single corner source model and average soil loading levels at both CEUS and WUS sites. Each profile was truncated at the bottom with hard basement rock with a Vs=2.83\ \sfrac{km}{sec} and Vs=1.0\ \sfrac{km}{sec} to account for the seismological conditions at CEUS and WUS sites, respectively. Eleven loading levels with a range of geological baserock peak accelerations from 0.01g to 1.50g were considered to cover a wide range of loading levels and accommodate nonlinear soil response.

Three sets of horizontal GDRS are selected based on a visual examination of the suites of median spectra. These are developed from results of site response analysis of each set of randomized profiles for both WUS and CEUS site conditions and for responses at the top of the profiles. The vertical GDRS are developed by multiplying the horizontal GDRS by frequency dependent V/H ratio appropriate for the selected site conditions. The GDRS define the design ground motion at the surface of in-situ soil.

7.3 Generic Profiles of Dynamic Subgrade Properties

Eight generic profiles define the following dynamic properties of subgrade materials as a function of depth for the BWRX-300 generic seismic design:

A. Total unit weight (w) representing best estimates of the combined weight of the saturated soil and the pore water

B. Strain-compatible shear wave velocities (V_S) shown on Figure 7-2

C. Poisson's ratio (ν) representative of saturated soil conditions shown on Figure 7-3

D. Compression wave velocities (V_P) shown on Figure 7-4

E. Strain-compatible damping shown on Figure 7-5

Stiffness and damping properties are compatible to the strains generated by a design level earthquake event. These strain-compatible properties are obtained from the results of the set of probabilistic site response analyses performed on the randomized base-case profiles of measured or small-strain V_S that were used to calculate the GDRS as described in Section 7.2.

The top softer soil layers in the SSI analysis subgrade profiles reflect saturated soil properties with values of ν ranging from 0.48 to 0.49. The maximum value of ν is kept below 0.49 to ensure the numerical stability of the SSI analysis results. The V_P profiles shown on Figure 7-4 are calculated from the following elastic theory equation based on the strain-compatible V_S and ν values:

$$V_P = V_S \sqrt{\frac{2(1-\nu)}{1-2\nu}} \tag{7-1}$$

The V_P profiles are representative of saturated soil conditions with the V_P of softer soil layers close to the value of water $V_P \approx 1600$ m/sec.

The generic profiles are categorized in terms of average measured small-strain shear wave velocity of the top 30 meters of soil ($\bar{V}s_{30}$) and the depths to the geological base-rock. For example, the generic profile 180-600 represents a generic site subgrade condition where the average measured (small-strain) shear wave velocity of top 30 m of soil $\bar{V}s_{30} = 180\ m/sec$ and the geological base-rock is located at depth approximately 600 m below the profile surface.

The eight generic profiles represent a range of generic site conditions varying from deep soft soil represented by Profile 180-600 to hard rock represented by Profile 2032-30 and cover a wide range of subgrade properties at the majority (>80%) of candidate sites.

A wide variation of shallow soil stiffness conditions is captured by considering profiles with:

- $\bar{V}s_{30} = 180\ m/sec$ representative of medium stiff soil sites
- $\bar{V}s_{30} = 270\ m/sec$ representative of firm soil sites
- $\bar{V}s_{30} = 400\ m/sec$ representative of stiff soil sites
- $\bar{V}s_{30} = 500\ m/sec$ and $760\ m/sec$ representative of soft rock sites
- $\bar{V}s_{30} = 900\ m/sec$ representative of firm rock sites

- $\bar{V}s_{30} = 2032\, m/sec$ representative of hard rock sites

A suite of profile depths to rock conditions are considered, ranging from approximately 8 m to 600 m, to accommodate the possible range in profile depths. The generic profiles consider soil removal, if necessary, to provide access to the site and conditions necessary for adequate support and operation of heavy cranes and other construction equipment.

The generic layered profiles reflect realistic site conditions with continuously increasing stiffness of the soil with depth due to confining pressure increase for the softer profiles (e.g., sands, gravels, loess, and till) and a decrease in weathering with increasing depth for the stiffer profiles (e.g., saprolite). A realistic non-monotonic change in strain compatible shear-wave velocity and damping with depth reflect the complex response of the soil shear columns. The nonuniformity of the strain compatible soil profiles increases the reflection of the waves propagating through the site and decreases the radial damping of the SSI system resulting in amplified peak SSI structural responses when the structural frequencies are resonant with the soil column frequencies.

For each generic profile, the nominal value of small-strain shear velocity of top 30 m of soil ($\bar{V}s_{30}$), the actual depth (H_S), the average shear wave velocity (V_{S_ave}), and shear column frequency (f_S) of the soil column above the base rock are provided in Table 7-1. The V_{S_ave} values in Table 7-1 are calculated using the equivalent arrival time method as follows:

$$V_{S\_ave} = {H_S}\Big/{\sum_i {d_i}/{V_{Si}}} \qquad (7\text{-}2)$$

where: d_i is thickness of each soil layer "i"

V_{Si} is the shear wave velocity of each soil layer "i"

The soil shear column frequency values listed in Table 7-1 are calculated as follows:

$$f_S = \frac{V_{S\_ave}}{4\, H_S} \qquad (7\text{-}3)$$

The profiles for generic design of BWRX-300 are developed as described in SMiRT-22 (Reference 8.59) based on a suite of base-case profiles selected from a database of measured (small-strain) subgrade properties. The base-case profiles were developed by averaging measured shear wave velocities for profiles with similar surficial geology or velocities. Because the number of available profiles generally decreases rapidly with depth, the profiles were extended to deeper depths considering typical geology for the considered type of sites.

Generic shear modulus reduction and hysteretic damping curves from EPRI TR-102293 "Guidelines for Determining Design Basis Ground Motions" (Reference 8.60) were used as input for the equivalent linear site response analyses to address the non-linearity of the soft rock and soil. The shear modulus reduction and hysteretic damping curves used for the soil materials are appropriate for gravels, sands, and low PI clays. The same suite of profiles and nonlinear dynamic soil properties were used to reflect conditions of CEUS and WUS sites.

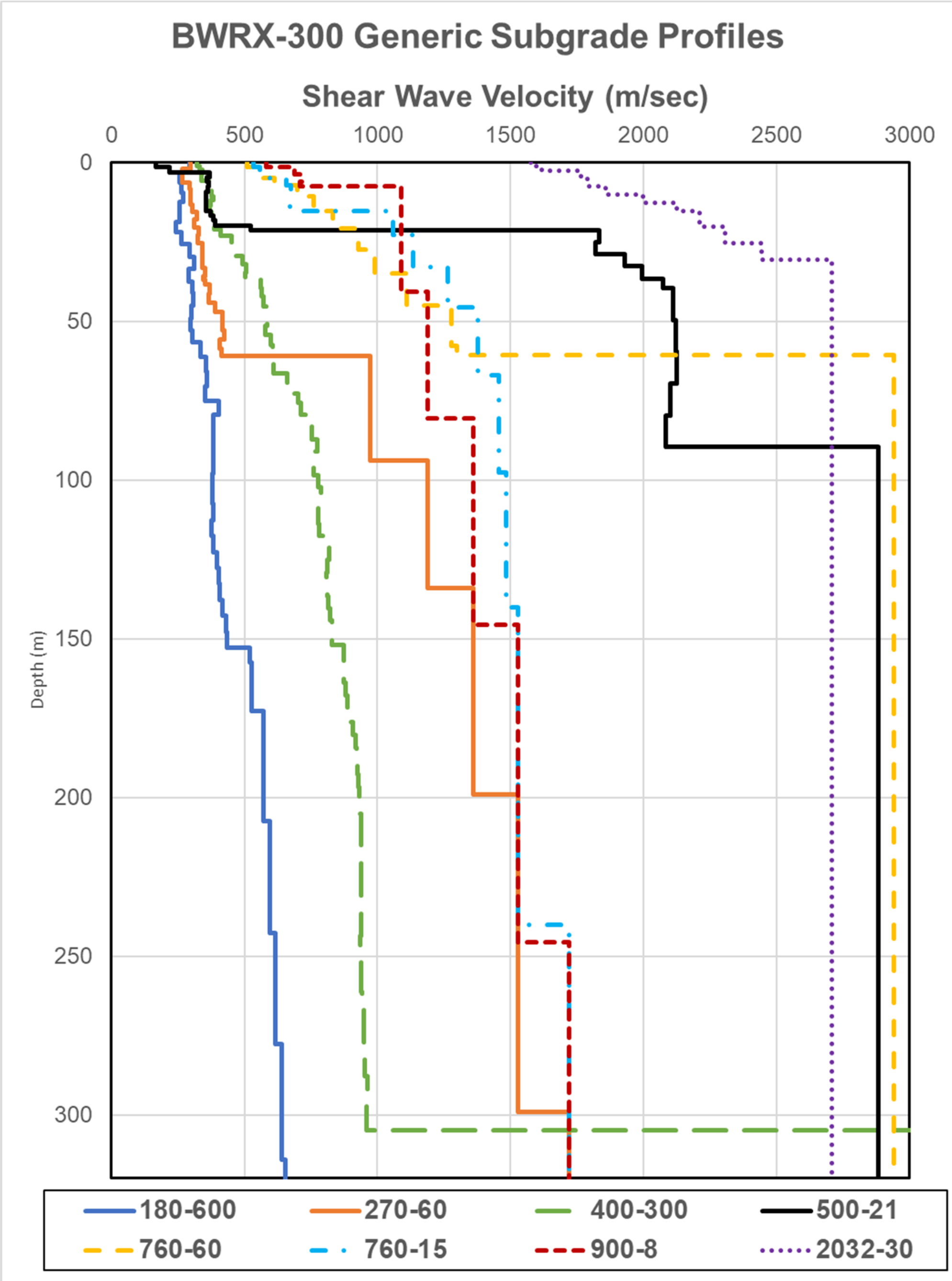


Figure 7-2: BWRX-300 Generic Shear Velocity Profiles

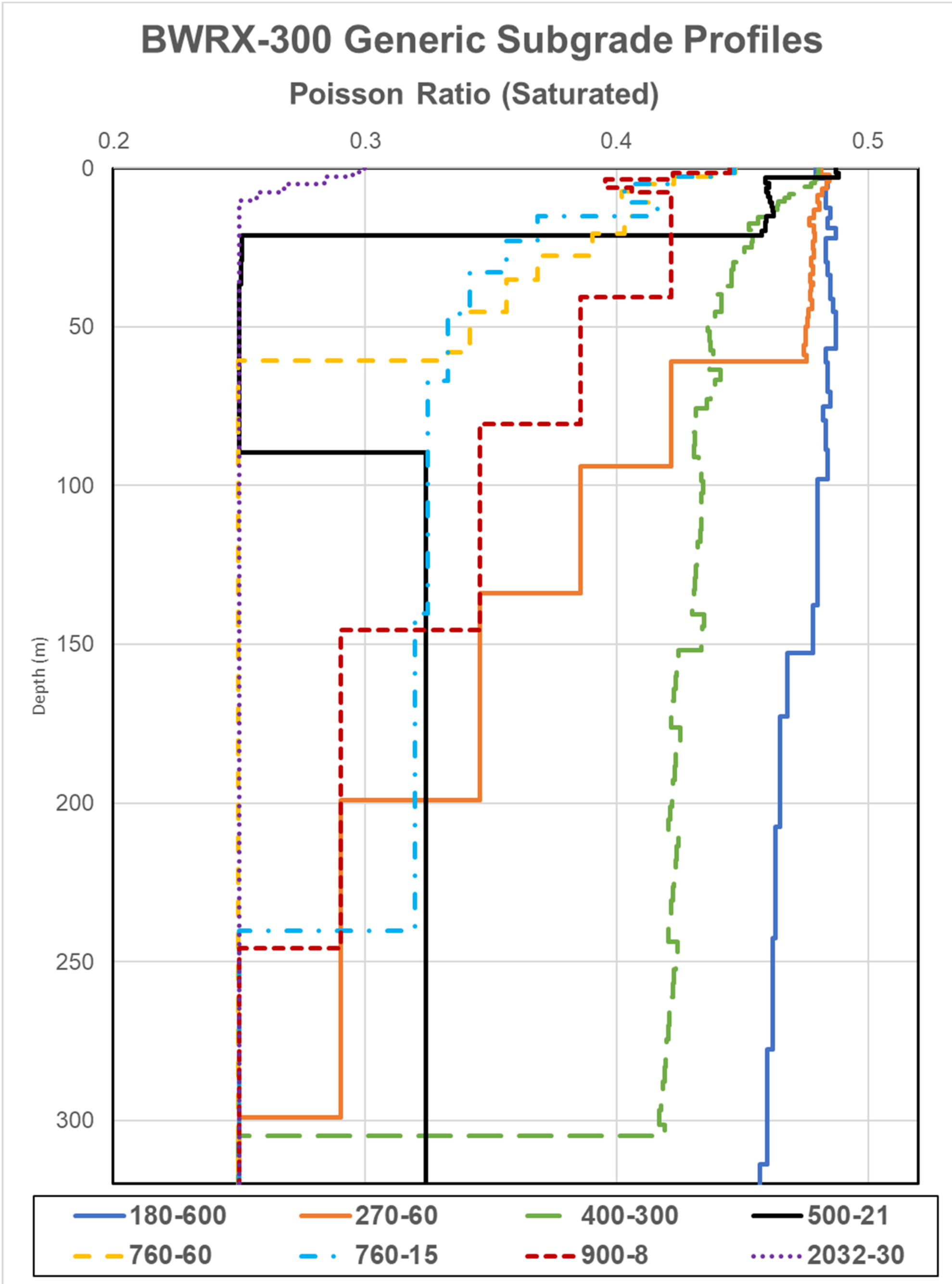


Figure 7-3: BWRX-300 Generic Saturated Soil Poisson Ratio Profiles

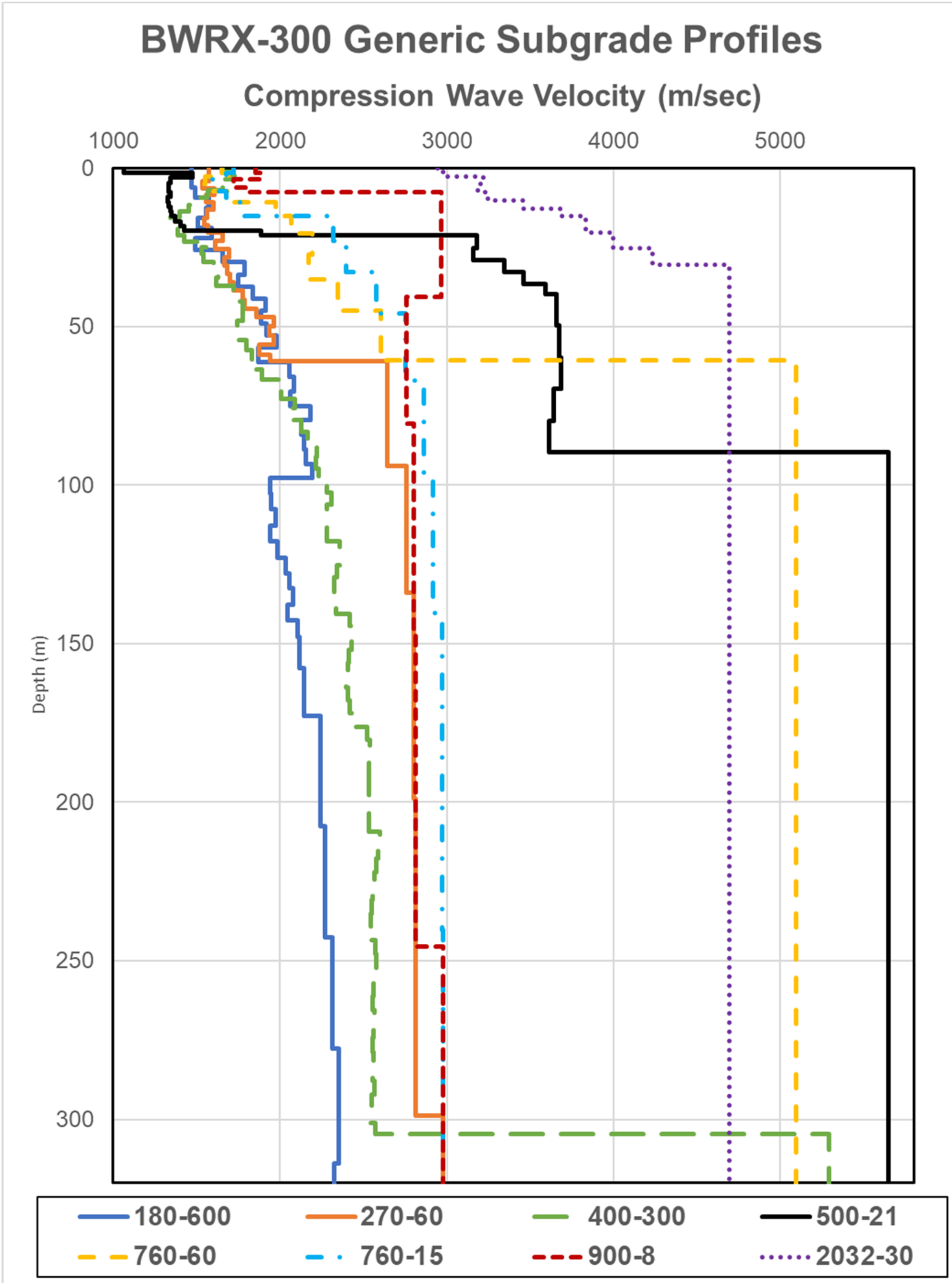


Figure 7-4: BWRX-300 Generic Compression Velocity Profiles

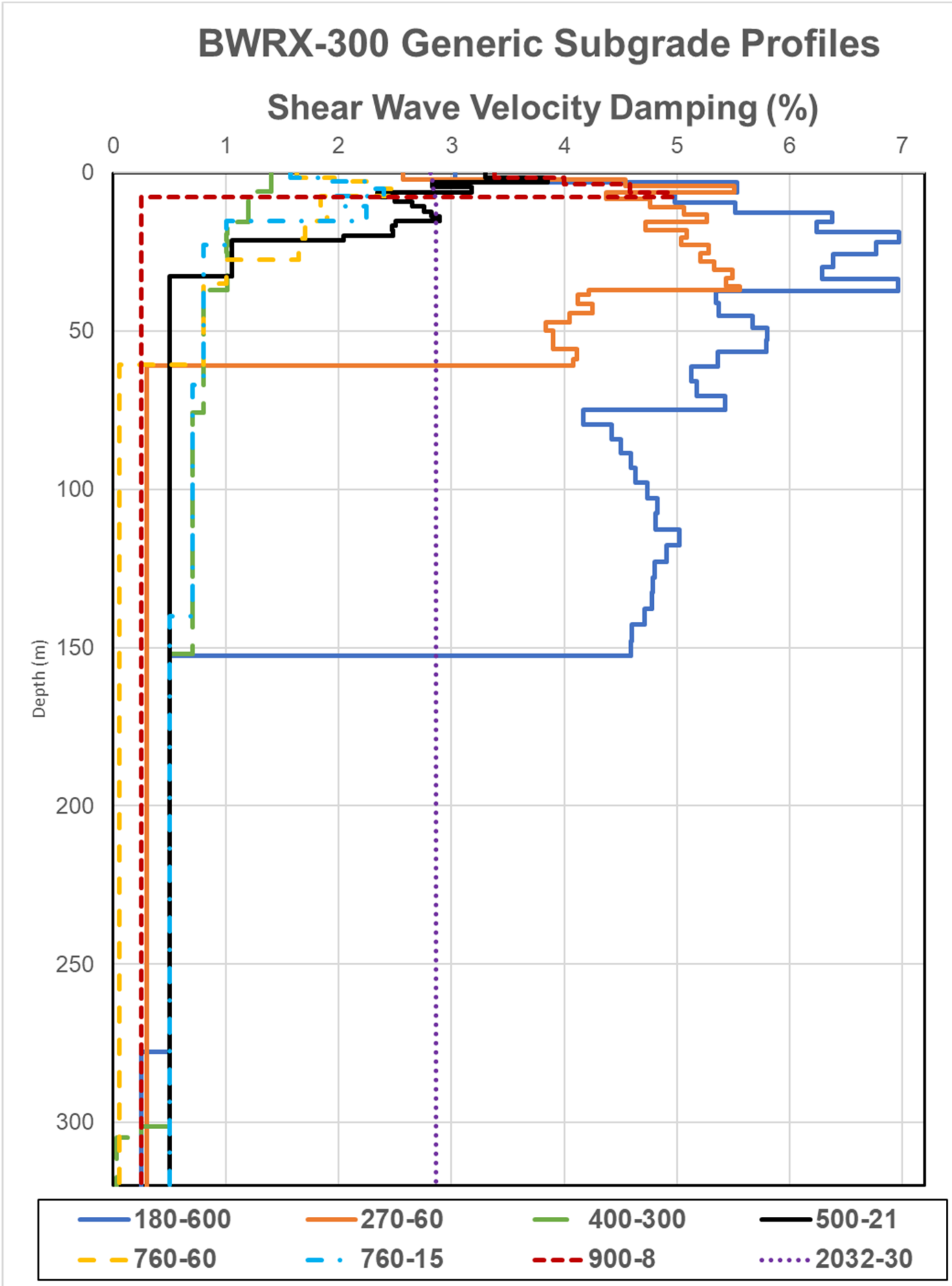


Figure 7-5: BWRX-300 Generic Damping Profiles

7.4 BWRX-300 Generic Design Soil Parameters

The following parameters for generic engineering properties of different types of subgrade materials at candidate nuclear sites used for the generic conceptual design of the BWRX-300:

- Dry and total unit weights (w_S)
- Range of void ratio values (e)
- Internal friction angle (ϕ_S)
- At-rest lateral pressure coefficient (K_0)
- Active lateral pressure coefficient (K_a)
- Passive lateral pressure coefficient (K_p)

The generic medium, firm, and stiff soil design parameters provided in Table 7-2, can be related to the generic profiles provided in Section 7.3 based on the $\bar{V}s_{30}$ nomination. The properties of loose and compact soil materials are also provided in Table 7-2 that are not adequate for supporting the BWRX-300 RB, RwB, TB and CB foundations that are usually removed from the site.

The values in Table 7-2 are based on the generic soil properties provided in the DOT design manuals (e.g., Iowa DOT 200E-1, Reference 8.61). The generic soil properties are based on the properties of cohesionless soil materials and are adequate representations for the soil conditions at the majority of candidate sites. The values for the soil unit weights and void ratios are calculated based on the information provided in Table 3.3 of "Soil Mechanics" (Reference 8.62) for silty sand and gravel subgrade materials. The use of silty sand and gravel subgrade soil properties, when in a well compacted state, is characterized by the highest unit weights among those of other granular soils, and ensures conservative upper bound values are calculated for the soil unit weights.

A range of values for the soil void ratio (e) provided in Table 7-2 for each soil material are calculated as follows, using the range of relative density values (D_r) provided in Table 2 of Iowa DOT 200E-1 (Reference 8.61):

$$e = e_{max}(1 - D_r) + D_r e_{min} \tag{7-4}$$

where: e_{max} is maximum void ratio

e_{min} is minimum void ratio

The maximum and minimum void ratio values are taken from Table 3.3 of Reference 8.62 for silty sand and gravel soils. Upper bound void ratio (e) value is calculated for each soil material in Table 7-2 using the lower bound values for the relative density (D_r) provided in Table 2 of Iowa DOT 200E-1 (Reference 8.61). Because the soil void ratio is directly related to the hydraulic conductivity of granular soil, these upper bound (e) values can be used as a conservative indicator for the water permeability of the subgrade materials for consideration of dewatering costs in the construction optimization studies.

The upper bound void (e) ratio values are directly related to the conservative unit weight values provided in Table 7-2 for use as input for the generic design calculations of load demands related to the soil weight. Conservative upper bound values for the soil dry unit weights are calculated as

follows using the upper bound values for the relative density (D_r) provided in Table 2 of Iowa DOT 200E-1 (Reference 8.61):

$$\text{Dry } w_s = \frac{w_{max} w_{min}}{w_{max}(1 - D_r) + D_r w_{min}} \tag{7-5}$$

where: w_{max} is maximum dry unit weight

w_{min} is minimum dry unit weight

Dry unit weight maximum and minimum values are taken from Table 3.3 of Reference 8.62 for silty sand and gravel soils.

Values for the total unit weight of generic soil materials are calculated as follows:

$$\text{Total } w_s = \text{Dry } w_s + \frac{e}{1+e} w_w \tag{7-6}$$

where: e is a lower bound value of the void ratio in Table 7-2

w_{min} is the unit weight of water

The values of dry unit weight and lateral pressure coefficients are used for calculation of lateral pressure demands. The values of lateral pressure coefficients are based on the lower bound strength properties of granular materials represented by generic values of internal friction coefficient (ϕ_s) provided in Table 7-2. Lower bound ϕ_s values are selected from the values provided for different soil materials in Table 2 of Iowa DOT 200E-1 (Reference 8.61).

At-rest lateral pressure coefficients (K_0) are provided in Table 7-2 for calculation of static lateral pressure loads on BWRX-300 below grade walls. Values of K_0 for soil materials are calculated using the Jacky's empirical equation shown in Equation (5-15). Table 7-2 provides a generic value of K_0 for rock that is calculated based on the following elastic theory equation using rock Poisson's ratio $\nu_r = 0.3$:

$$K_0 = \frac{\nu_r}{1 - \nu_r} \tag{7-7}$$

The soil active (K_a) and passive (K_p) lateral pressure coefficients in Table 7-2 are calculated using the following Rankine theory equations (Equations 13.1 and 13.2 in Reference 8.62):

$$K_a = \frac{1 - \sin(\phi_s)}{1 + \sin(\phi_s)}$$
$$K_p = \frac{1 + \sin\,(\phi_s)}{1 - \sin\,(\phi_s)} \tag{7-8}$$

Passive pressure coefficients (K_p) in Table 7-2 are used for the BWRX-300 generic design that provide a conservative estimate of the lateral bearing pressure capacity of the subgrade materials. The active pressure coefficients (K_a) listed in Table 7-2 can only be used for calculation of lateral pressure demands on soil retaining walls that are associated with larger lateral deformations.

Table 7-2: Generic Soil and Rock Parameters

Soil Type	Unit Weight (w_S) (kN/m³)		Void ratio (e)		Friction Angle ϕ_S (degree)	Lateral Pressure Coefficients			Base Friction Coefficient (μ_b) (1)
	Dry	Total	max	min		K_0	K_a	K_p	
Loose	16.6	20.1	0.708	0.566	30	0.671	0.333	3.00	0.36
Medium (Compact) Soil	18.3	21.2	0.556	0.424	35	0.620	0.271	3.69	0.43
Firm (Dense) Soil	20.4	22.5	0.424	0.282	40	0.568	0.217	4.60	0.50
Stiff (Very Dense) Soil	23.0	24.1	0.282	0.140	45	0.518	0.172	5.82	0.58
Rock	25.0	25.0	N/A		N/A	0.429	N/A	5.82 (2)	0.60

NOTES:

(1) If water proofing membrane is placed below basemat use minimum of provided value and 0.5

(2) Conservatively assumed value equal to the value calculated for the very dense soil

7.5 BWRX-300 Generic Profiles of Static Subgrade Properties

Eight generic profiles define the variation with depth of the following static properties of subgrade materials for the BWRX-300 generic design evaluations:

- Dry unit weight shown on Figure 7-6
- Soil solid phase Young's modulus (E) shown on Figure 7-7
- Soil solid phase Poisson's Ratio (ν_{st}) shown on Figure 7-8.

Profiles of generic subgrade static properties are developed to calculate conservative soil pressure demands on the BWRX-300 structures. The following criteria are used to correlate the generic design parameters in Table 7-2 to the small-strain V_S profiles that were used for the development of strain-compatible V_S profiles:

- Medium soil properties provided in Table 7-2 correlate to layers with small-strain V_S lower than 350 m/sec
- Firm soil properties provided in Table 7-2 correlate to layers with small-strain V_S ranging from 350 m/sec to 760 m/sec
- Stiff soil properties provided in Table 7-2 correlate to layers with small-strain V_S ranging from 760 m/sec to 1000 m/sec
- Rock properties provided in Table 7-2 correlate to layers with small-strain V_S larger than 1000 m/sec

As noted in Section 5.1.2, for the purpose of calculating lateral soil pressures, static analysis can neglect the weight of the rock layers with V_S larger than 1000 m/sec, considering them self- supporting and requiring no lateral support when excavated.

The theory of elasticity Equation (5-14) is used to calculate the ν_{st} representative of at rest lateral pressure conditions. Lower bound Young's modulus (E) values that only represent the stiffness of the soil solid phase are calculated using Equation (5-11) and a value of static stiffness degradation coefficient $D_E = 0.28$. This value for D_E is adopted based on the elastic modulus (E/E_0) degradation curve on Figure 8-15 of FHWA NHI 16 072: "Geotechnical Site Characterization" (Reference 8.63) considering the anticipated strain levels for the bearing capacity factor with a factor of safety of 3. The E/E_0 degradation in FHWA NHI-16-072 is applicable for intact clay and uncemented sand materials and provides lower values of stiffness degradation for the self supporting rock strata.

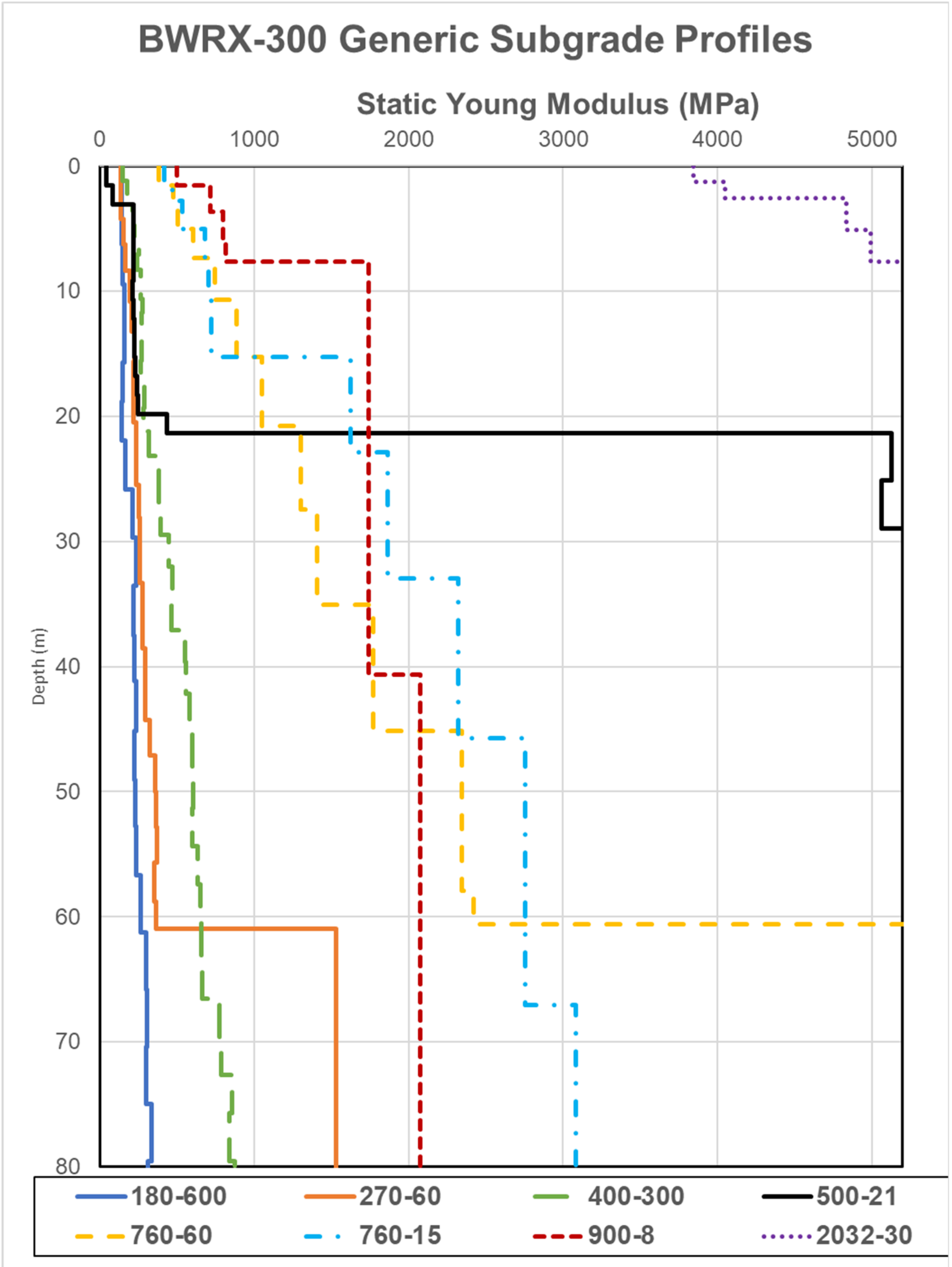


Figure 7-6: BWRX-300 Generic Dry Unit Weight Static Analysis Profiles

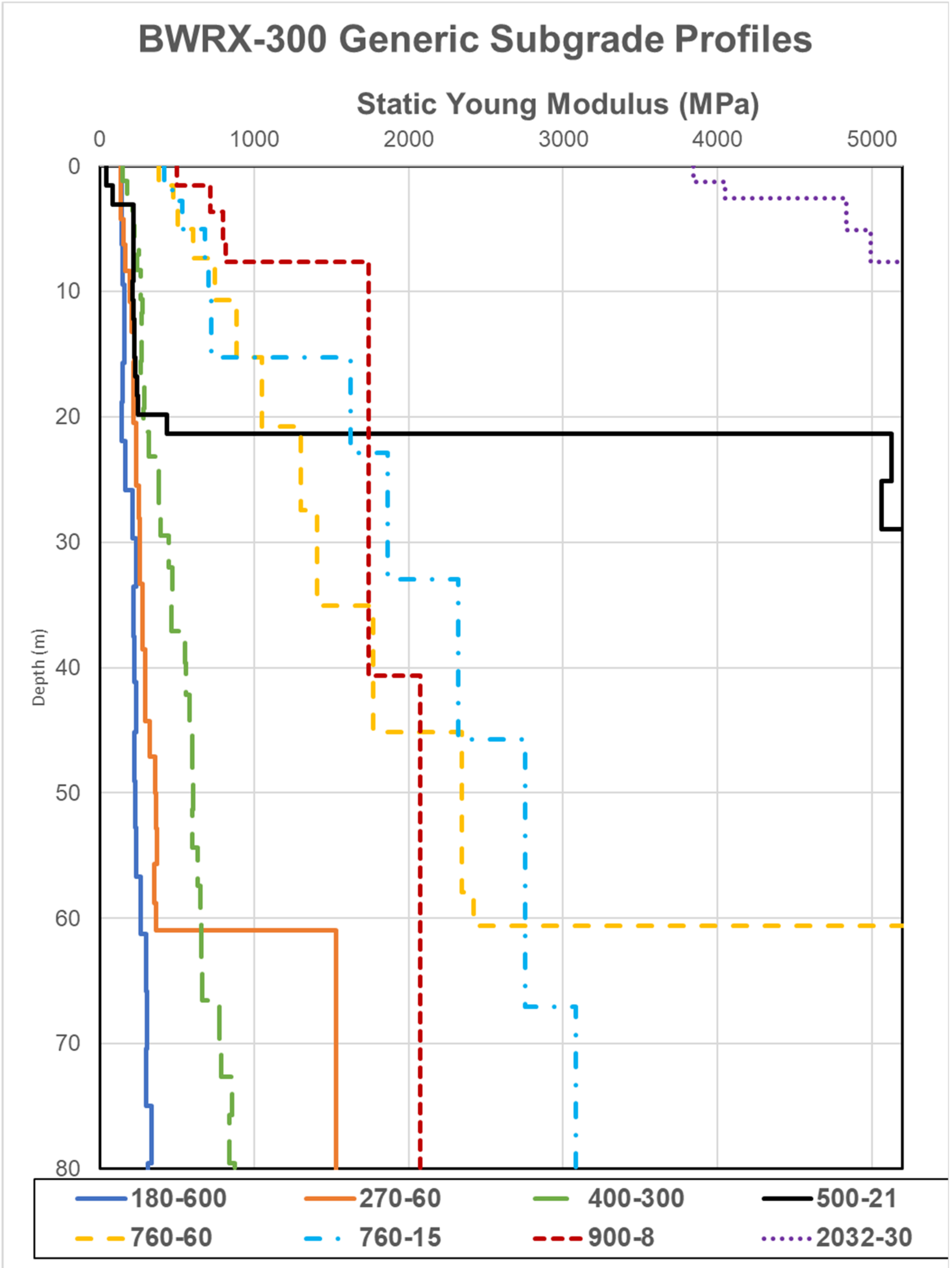


Figure 7-7: BWRX-300 Generic Young's Modulus Static Analysis Profiles

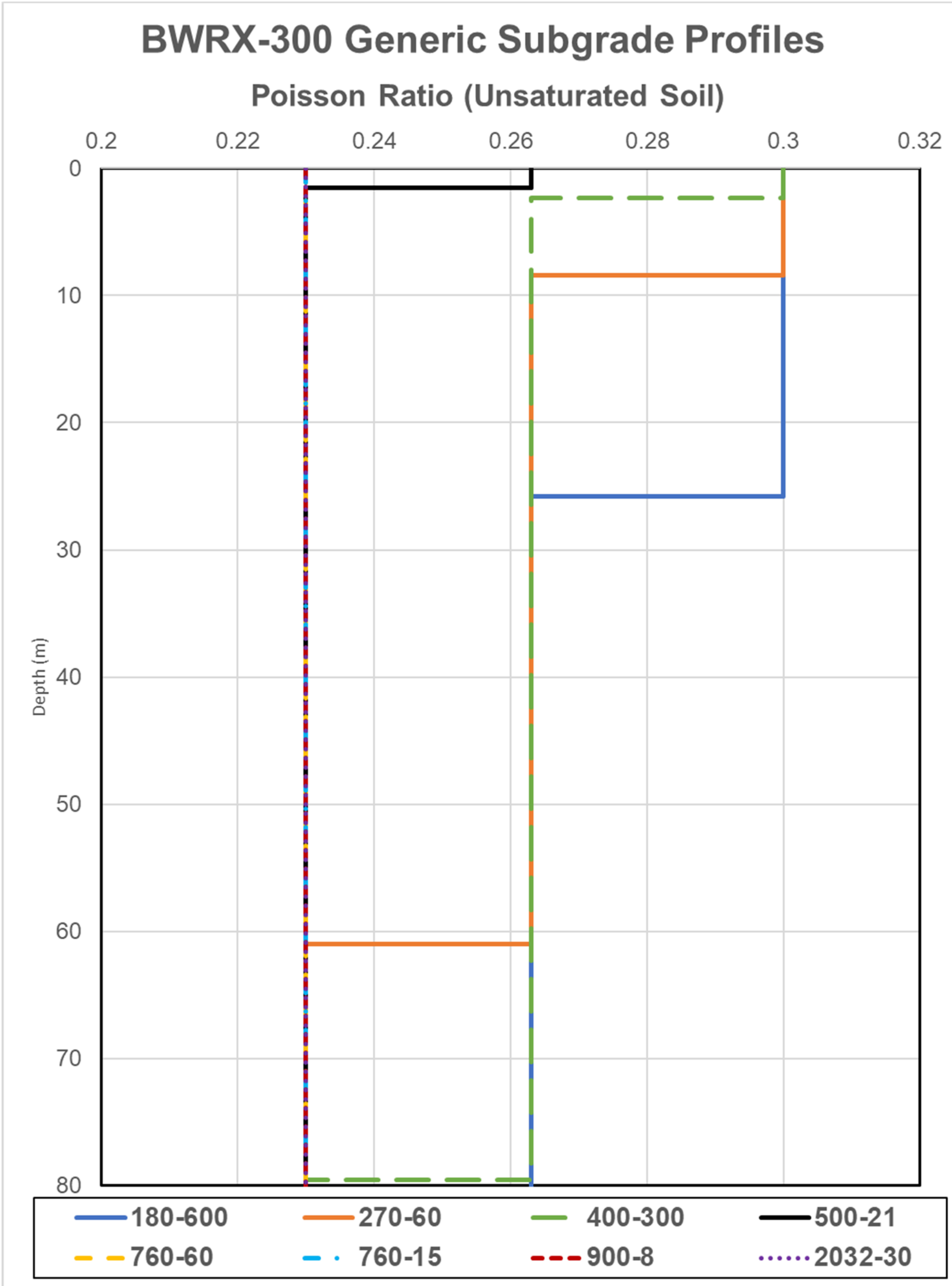


Figure 7-8: BWRX-300 Generic Dry Soil Poisson Ratio Static Analysis Profiles

7.6 BWRX-300 Generic Design Base Shear Friction Coefficients

The last column in Table 7-2 provides generic values for friction coefficients between the concrete basemats and different types of underlying subgrade materials for use as input for the BWRX-300 generic sliding stability evaluations. Based on common engineering practice, generic values for base friction coefficients (μ_b) are calculated as two-thirds (2/3) of the soil internal angle (ϕ_s). A value of 0.60 is adopted for the friction coefficient between the concrete basemat and underlaying rock based on engineering practice.

If a water proofing membrane is placed below the basemat, the generic design sliding stability evaluations shall use a minimum value of the provided soil base friction angle and membrane coefficient of friction with concrete. A conservative value of 0.5 for the membrane coefficient of friction is based on the results of testing of different water proofing materials.

7.7 BWRX-300 Generic Design Nominal Ground Water Level

The BWRX-300 generic design uses ground water pressure demands based on conservatively selected nominal groundwater level located at plant grade. The same groundwater level is used in the stability calculations to account for the buoyancy force.

The bearing pressure calculations and construction optimization evaluations are performed considering two bounding groundwater level elevations located at plant grade and below the BWRX-300 RB foundation bottom.

7.8 Summary of BWRX-300 Generic Design Approach

The following related to the methodology for development of generic site parameters for conceptual design of BWRX-300, presented in this section of the report, may be referenced during future licensing activities that are beyond the current guidance of SRP 2.0:

(1) The methodology for development of GDRS defining the design ground motion for the generic seismic design of BWRX-300, presented in Section 7.2.

(2) The methodology for development of eight generic profiles of dynamic subgrade properties for use as input for conceptual design SSI analyses of BWRX-300, presented in Section 7.3, representative of wide range of subgrade conditions present at the candidate sites.

(3) The methodologies for selection of parameters and development of profiles defining generic soil and rock properties for use as input for conceptual design static analysis of BWRX-300, presented in Sections 7.4 and 7.5, respectively.

(4) The methodology for selection of base shear friction coefficient values for use as input for conceptual design seismic stability evaluations of BWRX-300 foundations, presented in Section 7.6.

(5) The methodology for selection of groundwater elevations for generic design of BWRX-300, representing bounding groundwater conditions at most candidate sites.

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Appendix A
GEH Responses to NRC RAIs on NEDO-33914, Revision 0

SRP-Review Section: 01.05 - Other Regulatory Considerations
LTR Application Section: TR NEDO-33914 Sections 1.3 and 6.1.2

01.05-01 (eRAI 9849)
Date of eRAI Issue: 07/19/2021

Requirement

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

Regulatory Guide (RG) 1.143, "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed In Light-Water-Cooled Nuclear Power Plants," provides the guidance to licensees and applicants in the design, construction, installation, and testing the SSCs of radioactive waste management facilities in light-water-reactor nuclear power plants.

Issue

Sections 1.3 and 6.1 of GE-Hitachi Nuclear Energy Americas, LLC (GEH) Pre-Application Submittal of NEDO-33914, BWRX-300 Advanced Civil Construction and Design Approach Licensing Topical Report (LTR), submitted to the NRC on January 20, 2021 (Agencywide Documents Access and Management System (ADAMS) Accession No. ML21020A136) state that the portions of the turbine building structure and foundation that support and enclose the main steam piping are designed as RG 1.143, class RW-IIa. They also state that RG 1.143 is used because the building contain SSCs used for management and containment of highly radioactive gas, liquid, and solid materials whose failure, considering the maximum inventory, would result in a potential unmitigated radiological release levels that may be higher than those specified in RG 1.143, Section 5.1.

RG 1.143 provides guidance for the classification and design of radwaste management systems and steam generator blowdown systems. RG 1.143 does not provide guidance for the classification or design of the main steam piping or surrounding structures. While the offgas system is used for management of radioactive gas, other SSCs in the turbine building, like the main steam piping and the main condenser are credited for main steam line fission product holdup and retention in the analysis of design-basis accident radiological consequences for boiling water reactor plants with no main steam isolation valve leakage control system. In this way, the main steam piping and condenser are used to mitigate the consequences of an accident. Appendix A to 10 CFR Part 100 requires that SSCs necessary to ensure the capability to mitigate the consequences of accidents remain functional during and after a safe-shutdown earthquake (SSE).

RG 1.143 seismic classification of RW-IIa, specifies ½ (SSE) as the earthquake design criteria for radwaste management SSCs. If the ½ SSE design requirement is applied to the condenser and portions of the main steam piping in the turbine building, the capability for those systems to mitigate the consequences of accidents and remain functional during and after an SSE would not be ensured.

Request

The staff requests GEH to clearly identify the applicability of RG 1.143 to the turbine building design. The response should also address any limitations on the applicability of RG 1.143 and clarify how the design will ensure SSCs in the turbine building that are used to mitigate the consequences of an accident be designed to remain functional during and after an SSE.

GEH Response to NRC Question 01.05-01

In the initial submittal of NEDO-33914, the decision to designate this portion of the turbine building (TB) as RW-IIa was associated with the offgas system (OGS) charcoal adsorbers that were located in the same area as the main steam piping. The OGS charcoal adsorbers are used for management of radiological gases are designated as RW-IIa, as required by the guidance in RG 1.143, C. Regulatory Position, 5. Classification of Radwaste Systems for Design Purposes, and 6. Natural Phenomena and Man-Induced Hazards Design for Radwaste Management Systems and Structures.

GEH has elected to move the OGS charcoal adsorbers to the radioactive waste building (RwB), thereby eliminating the need for the associated portion of the TB to comply with RG 1.143 "Design Guidance For Radioactive Waste Management Systems, Structures, And Components Installed in Light-Water-Cooled Nuclear Power Plants," and to be qualified as RW-IIa structure one-half safe shutdown earthquake (SSE). As a result, Sections 1.3, 2.4, 6.1 and 6.4 of licensing topical report (LTR) NEDO-33914 "BWRX-300 Advanced Civil Construction and Design Approach" are being revised to reflect the relocation of the OGS charcoal adsorbers to the RwB. The remaining equipment in the TB associated with the OGS such as the offgas cooler and refrigerant dryer are processing equipment that do not hold up or accumulate gaseous effluents, and are not required to meet the classification of RG 1.143 RW-IIa or RW-IIb classification. Therefore, the entire TB is now classified as non-seismic as described in LTR NEDO-33914, Sections 6.1 and 6.4. The II/I seismic interaction evaluation of the TB is described in LTR NEDO-33914, Section 6.2.

Regarding the second response request to clarify how the SSCs in the turbine building that are used to mitigate the consequences of an accident be designed to remain functional during and after an SSE, GEH offers the following response:

Like the Advanced Boiling Water Reactor (ABWR) and Economic Simplified Boiling Water Reactor (ESBWR), the BWRX-300 design safety analyses confirm that there are no design basis accidents (DBAs) that result in fuel damage. As a result, the main steam line and condenser are not credited for fission product holdup or retention of any radiological consequences.

The BWRX-300 has further refined the barrier design compared to the ABWR and the ESBWR by incorporating the use of reactor pressure vessel isolation valves attached directly to the reactor pressure vessel (RPV). These RPV isolation valves also function as inside containment isolation valves (CIVs), along with one outboard main steam isolation valve (MSIV) that are used to preserve reactor coolant system inventory for large and medium pipe break loss-of-coolant-accidents (LOCAs) and to prevent releases from containment. This configuration was approved for use in accordance with 10 CFR 50, Appendix A, General Design Considerations (GDC), 55 Reactor coolant pressure boundary penetrating containment, in the final safety evaluation for GEH LTR NEDC-33911P, Revision 0, Supplement 1, "BWRX-300 Containment Performance" dated March 12, 2021, Section 5.1.22, "10 CFR Part 50, Appendix A, GDC 55". NEDC-33911P-A, Revision 1, Section 2.2.7.1, states that the automatic CIVs outside containment are not required to be fast closing because there is no credible scenario in which fission products can be released to the containment within a few hours of a DBA. As a result, there are no credible DBAs where postulated fission product releases greater than those contained in normal reactor coolant could occur for small-, medium-, or large-break LOCAs. CIV closure times will be established based upon source term evaluations for postulated beyond design basis accident fission product releases, and closure times will be ensured by compliance to the limiting conditions for operation in the BWRX-300 technical specifications. In the final safety evaluation for NEDC-33911P-A, Section 5.3.12, the NRC staff agreed with GEH's statement in Section 2.2.7.1 by stating: "the NRC staff finds the above clarification on evaluating outboard CIV closure time acceptable, because valve closure time is to be established based on fission product release and source term evaluation, consistent with the guidance in SRP Section 6.2.4, Paragraph I, Item 1.E regarding the basis for selection of closure times of isolation valves."

NEDC-33911P-A, Revision 1, Section 2.2.7, states that leak-tightness of the CIVs is verified by 10 CFR Part 50, Appendix J "Primary Reactor Containment Leakage Testing for Water-Cooled Power Reactors", Type C tests, and the leak-tightness of containment is verified by Type A testing. In NEDC-33911P-A, Revision 1, Section 5.2.7, GEH states that the design will include a containment leak test program that addresses integrated containment leakage rate (Type A tests), containment penetration leakage test (Type B tests) and CIV leakage rates (Type C tests) that complies with 10 CFR Part 50, Appendix J. Type A, B, and C tests are performed before operations and periodically thereafter to assure that the leakage rates through containment and through systems or components that penetrate containment do not exceed their maximum allowable rates. Maintenance of containment, including repairs on systems and components penetrating containment is performed as necessary to maintain leakage rates at or below acceptable values.

In the final safety evaluation for NEDC-33911P-A, Section 5.1.26, the NRC staff concluded: "The NRC staff finds the BWRX-300 design, as described in NEDC-33911P, would accommodate periodic integrated leakage rate testing and local leak rate test for CIVs, and containment penetrations; thus, the NRC staff finds that the design is consistent with 10 CFR Part 50, Appendix J and, therefore, is acceptable."

In the final safety evaluation for GEH LTR NEDC-33910P, Revision 0, "BWRX-300 Reactor Pressure Vessel Isolation and Overpressure Protection" dated November 18, 2020, Section 2.2, the

NRC staff agreed with the robustness of the RPV isolation valve concept: "The NRC staff finds that, based on the description in NEDC-33910, together with its reference to NEDC-33911, the RPV isolation valve concept is consistent with 10 CFR Part 50, Appendix A, GDC 30, "Quality of reactor coolant pressure boundary," and GDC 35 and is therefore acceptable." Further in the final safety evaluation for GEH LTR NEDC-33910P, Revision 0, Section 2.4.1, "Connection of the Reactor Pressure Vessel Isolation Valves to Reactor Vessel," the NRC staff further agreed with the robustness of the RPV isolations where they stated: "details of the bolted connections including threaded fastener design, leakage detection systems design, and ISI requirements, will demonstrate that the probability of gross rupture is extremely low. Section 2.4.1 specifies that these design and ISI requirements will provide assurance that potential failure mechanisms are detected before the onset of a catastrophic failure involving the fasteners of the bolted flange connections for the RPV isolation valves. Therefore, Section 2.4.1 asserts that a break at these locations need not be postulated."

Section 2.7 of NEDC-33910P-A, Revision 2, states that two categories of break sizes are specified in the topical report that differentiates between large pipes with isolation valves and smaller pipes without isolation valves (e.g., the differential pressure instrument lines). The key concept related to meeting the design requirements is that larger isolation lines are rapidly isolated to minimize inventory loss in conjunction with cooling provided by the isolation condenser system to maintain reactor water level at or above the top of active fuel (TAF) or fuel cladding temperature within normal operating temperature range for at least 72 hours. For smaller lines that can remain unisolated, the differential pressure between the RPV and the containment is minimized such that inventory loss is reduced in the absence of injection or makeup flow and core cooling is provided by the isolation condenser system (ICS) such that the reactor water level is maintained at or above the TAF or the fuel cladding temperature is maintained within normal operating range for at least 72 hours. In the final safety evaluation for GEH LTR NEDC-33910P, Revision 0, Section 2.7 Categories of Pipe Breaks, the NRC staff agreed with GEH's conclusion regarding pipe breaks: "The NRC staff finds that the categories of pipe breaks and the associated design requirements for the BWRX-300, as described in NEDC-33910, are consistent with 10 CFR 50.46(b)."

GEH has not determined that credit for holdup or retention of radionuclides in the main steam lines or the condenser for mitigating the consequences of beyond design basis accidents (BDBAs)/severe accidents (SAs) is necessary; however, based upon the robustness of the BWRX-300 RPV isolation valves and the outboard MSIV configuration that rely upon Class 1E battery-backed direct current power for automatic actuation and fail-safe design, and the stringent Appendix J leakage testing program, GEH's current analysis has not found the necessity of this credit. Therefore, the TB does not need to be designed to meet more stringent seismic requirements as the main steam lines and the condenser have not been credited for radiological consequences mitigation.

Additionally, GEH notes that the NRC staff has concluded that the design of the non-seismic BWR turbine buildings are robust as discussed in DRA-ISG-2021-XX, "Supplemental Guidance for Radiological Consequences Analyses Using Alternative Source Terms Draft Interim Staff Guidance," May 2021. In DRA-ISG-2021-XX, the NRC staff discusses the approved alternate pathway of the main steam piping downstream of the outboard MSIV and the main condenser that

was justified by the design information and data collected in GE topical report NEDC-31858P, Revision 2, "BWROG Report for Increasing MSIV Leakage Limits and Elimination of Leakage Control Systems," September 1993, ADAMS Accession No. ML010640286, and approved by U.S. Nuclear Regulatory Commission, "Safety Evaluation of GE Topical Report, NEDC-31858P, Revision 2, 'BWROG Report for Increasing MSIV Leakage Limits and Elimination of Leakage Control Systems,' September 1993," March 3, 1999, ADAMS Accession No. ML010640286. The NRC staff notes that GE has demonstrated that the BWR turbine building is "seismically robust" at the plant's SSE and the alternate pathway for crediting holdup and retention in the main steam line from the outboard MSIV is acceptable. The NRC staff notes: "This ISG provides supplemental guidance to items III.6.c and IV.5 in SRP Section 15.0.1. The basis for the supplemental guidance is a technical assessment that uses knowledge and operating experience related to the power conversion system (PCS), including information on the seismic capacity and risk at nuclear power plants. The staff, through this ISG, should acknowledge the presence of the PCS and its ability to provide a large holdup and retention volume for MSIV leakage when staff determines that the requirements of the regulations are satisfied and the method of analysis conforms with accepted practices, but uncertainties remain in input parameters used in the deterministic dose calculations. In doing so, the staff should recognize that there is a high probability that doses will be lower than those estimated using deterministic methods that include accepted assumptions but do not credit holdup and retention of the MSIV leakage within the PCS." This assessment uses engineering information, such as operations and design knowledge, and probabilistic and risk information on the seismic capacity (i.e., the ability of SSCs) to withstand acceleration induced by a seismic event) of the SSCs in the realistic transport pathway to determine the risk of unavailability of the SSCs in the PCS pathway for fission product holdup and retention. The safety evaluation on NEDC-31858P, Revision 2, gives precedent for not relying on only safety-related or seismic Category I SSCs for mitigating the radiological consequences of a postulated release. That safety evaluation states that requiring the non-seismically analyzed portions of the main steam system piping and components to meet seismic Category I requirements is impractical because the modifications required to upgrade the system to seismic Category I requirements would be very costly. This interim guidance concludes that BWR turbine buildings are "seismically robust" to SSEs and further substantiates that the BWRX-300 TB does not require more stringent SSE compliance.

Proposed Changes to NEDO-33914 Revision 0

NEDO-33914, Sections 1.3, 2.4, 6.1, 6.1.2 and 6.4 will be revised to reflect the relocation of the OGS charcoal adsorbers to the RwB. The RwB structure and foundation basis remains the same.

1.3 Description of the BWRX-300

……….

TB encloses the turbine generator, main condenser, condensate, and feedwater systems, condensate purification system, off-gas system (OGS) cooler and refrigerant dryer, turbine-generator support systems and bridge crane.

The RwB, which houses the systems for management of radioactive gas, liquid, and solid radiological waste is categorized as RW-IIa in accordance with Regulatory Guide (RG) 1.143 "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants," Revision 2. ~~The portions of the TB structure and foundation that support and enclose the main steam piping and the OGS for management of radiological gases are also designed as Rw-IIa following the provisions of RG 1.143.~~

2.4 II/I Interaction Regulations

………

- ~~The TB structure does not collapse to result in impairment of safety functions of the main steam piping or the OGS.~~

6.1 Control Building, Turbine Building and Radwaste Building Design Bases

CB, TB and RwB structures and foundations are designed in accordance with their seismic classification:

- Non Seismic Category for the CB and TB structures ~~excluding the portion of TB enclosing the main steam piping and offgas system (OGS)~~; and
- RW-IIa Category for the RwB structure ~~and the portion of TB structure enclosing the main steam piping and OGS~~.

6.1.2 Radwaste Category IIa Building Structure and Foundations Design Bases

RwB ~~and the portion of the TB enclosing the main steam piping and OGS~~ is classified as a RW-IIa structure~~s~~ because it contains SSCs used for managing and containment of highly radioactive gas, liquid, and solid materials whose failure, considering the maximum inventory, would result in a potential unmitigated radiological release levels that may be higher than those specified in RG 1.143, Section 5.1.

In accordance with RG 1.143, Table 1 guidance, the design of the BWRX-300 RwB steel structures follows the provisions of AISC N690 (Reference 8.25). The design of the RwB concrete structures and basemat ~~and the portion of the TB structure enclosing the main steam piping and OGS~~ is in accordance with ACI 349-13 (Reference 8.24). Based on RG 1.143, Table 2, the loads for the design of the RW-IIa RwB structure ~~and TB~~ includes:

6.4 Summary of Design Approach for II/I Interaction

The following aspects of the BWRX 300 graded design approach for II/I interaction of non-SC-I CB, TB and RwB with adjacent SC-I RB presented in this section of the report may be referenced

during future licensing activities that are beyond the current guidelines in RG 1.29, "Seismic Design Classification," Revision 5:

(1) General criteria for design of Non-Seismic CB and TB structures provided in Section 6.1.1, including the requirements for determining seismic and wind design loads.

(2) General criteria for design of RW-IIa, RwB ~~and TB~~ structures provided in Section 6.1.2, including the requirements for determining seismic, wind, tornado wind and missile design loads.

SRP Review Section: 02.05.04 - Stability of Subsurface Materials and Foundations

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02.05.04-01 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Sections: TR NEDO-33914 Sections 3.1.1, 3.1.2, 3.1.3, and 4.3.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Issue:

Figure 4-2 shows the rheological model of an interface to be used in the Foundation Interface Analysis (FIA) with several parameters, such as, k_n, σ_t, τ_{max}, k_s, C, C_r, φ. These parameters determine whether the interface slides (shear failure) or dilates (tensile failure) under the imposed loads including the load from the safe shutdown earthquake (SSE). The response of both soil and rock media surrounding the Reactor Building (RB) shaft to the imposed loads significantly affects the loads imposed on the RB walls. In addition, the loads imposed on the RB wall may not be symmetric around the shaft walls, especially in the rock medium.

Section 4.3.1.2, Fault or Joint Planes or Interfaces Between Bedding Units in a Geologic Formation, states that the nonlinearity and behavior of the joints are analyzed throughout the life stages of a reactor and the same interface model would be used in modeling the joints, bedding planes, and faults in the rock mass as part of the FIA model. The properties assigned to the interface elements along a rock discontinuity are to be obtained from laboratory or field testing (Sections 3.1.1 and 3.1.2). In addition, Section 4.3.1.2 states that the parameters representing slip of the interface model may be estimated based on properties of the weakest interface materials.

It is not clear from the discussions given in Site Investigation Program (Section 3.1.1) and Laboratory Testing Program (Section 3.1.2) whether a specific program would be developed to collect the necessary samples at the site and conduct specific tests at the laboratory to determine the parameters of the FIA model, as shown in Figure 4-2, or any other model to be used to represent

the interfaces. It is also not clear how the weakest plane (interface) would be identified at a given site with its strength properties.

Request:

The staff requests GEH to identify the sample collection and testing programs that would be used to determine the parameters necessary to model the behavior of all interfaces (RB Wall/Soil, RB Wall/Rock, Soil/Rock, and Rock/Rock for joints/bedding planes), as appropriate. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-01

The sample collection and testing program for each site will be based on the presumed subsurface conditions. Depending on the selected software used for the FIA model, the sample collection and testing program will be adjusted to characterize the soil and rock parameters. Figure 4-2 in Licensing Topical Report (LTR) NEDO-33914 provides example interface models that could be used for the BWRX-300 FIA model. Interface models will be selected considering the specific site conditions and analysis requirements. NEDO-33914 provides the general design approach and appropriate guidance to develop these plans and programs following Regulatory Guide (RG) 1.132 (Reference 01-1) and RG 1.138 (Reference 01-2).

The site investigation and laboratory testing program to determine the parameters necessary to model the behavior of soil, rock, and interfaces is based on Sections 3.1.1 and Section 3.1.2 of NEDO-33914. This section provides a list of minimum laboratory tests that need to be performed to develop subgrade properties for soil and rock.

Specific tests for interfaces are not identified; however, Section 3.1.3 of NEDO-33914 provides more detailed guidance on the evaluation of discontinuities from recovered rock samples and testing weaker infill materials. The details include characterization of the shear strength for planar discontinuities following RG 1.132 (Reference 01-1) on recovered samples and in situ tests. Corrections for surface roughness, intact surface strength, and sample scale are identified, and evaluation of infill in rock joints that may control the shear strength or deformation properties are discussed.

Section 4.3.1.1 of NEDO-33914 describes how the interface properties between the RB structure and the subgrade media may be determined and evaluated. This includes use of the adjacent soil/rock material for defining the interfaces and use of a strength reduction factor based on the roughness of interaction and soil/rock residual strengths when sliding occurs. The use of geosynthetic products at the soil/structure interface is addressed, and evaluation of uncertainty in the interface parameters using bounding estimates and sensitivity evaluations is described.

As appropriate for the subsurface conditions at a site, the selected FIA software, and the selected interface models, strength and deformation testing (e.g., triaxial and shear tests) may be completed on undisturbed or recompacted soil samples and recovered rock cores to understand the properties of these materials. These tests may also be performed on recovered joints and bedding planes to estimate the properties of those interfaces. Previous studies (e.g., Barton, 1972, Reference 01-3)

provide guidance on methods to estimate the deformation properties of joints based on laboratory tests and estimated properties of the rock mass.

Besides tests on recovered discontinuities and infill materials, tests on saw cut portions of intact rock cores or clean discontinuities may also be completed to evaluate the range of values associated with rock materials as discussed in EM 1110-1-2908 (Reference 01-4). This range of values would be considered to establish an estimate of the weakest discontinuity at a site and appropriate parameters for the rock. Testing on artificial discontinuities, like rock-structure interfaces, may also be completed (e.g., American Society for Testing and Materials (ASTM) D5607, Reference 01-5). In-situ direct shear tests on joints or bedding planes exposed at nearby outcrops or in shallow rock excavations (e.g., ASTM D4554, Reference 01-6; RTH 321-80, Reference 01-7) may be considered for some sites during the Site Investigation Program; however, these tests are less common and typically used when representative samples cannot be obtained.

References

01-1. RG 1.132, "Site Characterization Investigations for Nuclear Power Plants," Revision 2, October 2003.

01-2. RG 1.138, "Laboratory Investigations of Soil and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, December 2014.

01-3. Barton, N.R., "A Model Study of Rock-Joint Deformation," International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, Vol. 9, pp 579-602, 1972.

01-4. USACE Engineer Manual EM 1110-1-2908, "Rock Foundations," U.S. Army Corps of Engineers, Washington, DC, November 1994.

01-5. ASTM D5607 "Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force," 2016.

01-6. ASTM D4554 "Standard Test Method for In Situ Determination of Direct Shear Strength of Rock Discontinuities," 2012.

01-7. U.S. Army Corps of Engineers, "Suggested Method for In Situ Determination of Direct Shear Strength (ISRM)," RTH 321-80, Waterways Experiment Station, Vicksburg, MS, 1980.

Proposed Changes to NEDO-33914 Revision 0

Section 3.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to indicate testing of discontinuities should be included using the methods identified in RG 1.138 and that interface testing or evaluation of the interfaces as described in Section 4.3.1.1 can be considered. Additionally, direct shear tests are added to the minimum laboratory tests required for rock materials.

Section 3.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to identify direct shear and triaxial compressive tests as examples of laboratory testing on recovered samples; identify testing on sawn portions of recovered rock cores to determine base friction angles; and, identify in-situ direct shear tests as an option for shallow rock excavations.

Section 4.3.1.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to indicate the parameters from adjacent soil/rock elements can be based on the results from the Site Investigation and Laboratory Testing program.

References in NEDO-33914 are added for RG 1.132 and RG 1.138.

02.05.04-02 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Sections: TR NEDO-33914 Sections 3.1.1, 3.1.3, 3.2.1, 4.2.2, and 5.2.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Revision 2, describes methods acceptable to the NRC staff for conducting field investigations to acquire the geological and engineering characteristics of the site and provides recommendations for developing site-specific guidance for conducting subsurface investigations.

RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, describes laboratory investigations and testing practices for determining soil and rock properties and characteristics needed for engineering analysis and design of foundations and earthworks for nuclear power plants.

Issue:

It is not clear from the discussion given in Section 3.1.3, Characterization of Rock Mass Properties, whether the geological characterization of the rock unit(s) would be adequate to represent the rock mass in the FIA analyses. Discussions on fracture network characterization of the rock mass is mostly limited to collecting 1D information through boreholes. However, rock fractures are 3D in nature and occur in sets (joint sets). Multiple sets of rock joints can be present in a rock mass creating individual rock blocks. Additionally, the rock mass may be a bedded deposit comprising of multiple rock beds. No discussion is given in Section 3.1.3 how the rock mass fracture network, which can significantly influence the rock pressure of the RB walls, would be characterized.

Additionally, Section 3.1.3, Characterization of Rock Mass Properties, discusses the use of rock mass classification systems (e.g., the Rock Mass Rating (RMR) system, the Geological Strength Index (GSI) system) to develop an estimate of the stress-strain behavior of rock (Section 4.2.2, Rock Constitutive Model) and rock mass stiffness properties (Section 5.2.1.2, Rock Mass

Equivalent Linear Properties). The RMR system specifically requires information of the rock discontinuity spacing, orientation, and conditions. The GSI system requires information on at least J_r (joint roughness number) and J_a (joint alteration number) parameters to determine the specific GSI value of the rock mass. It is not clear how these parameters would be determined based on discussion given in Section 3.1.1, Site Investigation Program. Additionally, it is not clear what inspection and verification programs would be used during the Construction Phase (Section 3.2.1, Excavation and Foundation Inspections and Testing) to verify the assumptions made about the rock mass (e.g., rock fracture network, joint strength, etc.) before the excavation commences.

Request:

The staff requests GEH to identify the plan(s) and program(s) for characterizing the rock fracture network and determining the necessary parameters for the rock mass classification system used to determine the rock mass stress-strain behavior. The staff is also requesting GEH to identify the program(s) to verify the assumptions made of the rock and soil media surrounding the RB shaft as the excavation progresses. Modify the TR, as necessary.

GEH Response to NRC Question 02.05.04-02

The plan(s) and program(s) for characterizing the rock fracture network and rock mass classification system for each site will need to be based on the presumed subsurface conditions and the selected rock mass classification system. The Licensing Topical Report (LTR) NEDO-33914 provides the general design approach and appropriate guidelines to develop these plans and programs following Regulatory Guide (RG) 1.132 (Reference 02-1) and RG 1.138 (Reference 02-2).

Section 3.1.3 of NEDO-33914 identifies multiple empirical engineering and geo-mechanical rock mass classifications, such as the Rock Quality Designation (RQD) index, the Rock Tunneling Quality (Q) index, the 1976 and 1989 versions of the Rock Mass Rating (RMR) system, and the Geological Strength Index (GSI), that may be used. Additional details are provided in the same section on the RMR system and GSI.

Estimation of the parameters for the RMR and GSI systems may be made using the data from the geotechnical borings, wells, and borehole televiewer locations as discussed in Sections 3.1.1 and 3.1.3 of NEDO-33914. The geotechnical borings can provide recovered cores for estimating RMR and GSI parameters. Inputs for estimating these parameters will include:

- Uniaxial compressive strength and triaxial strength testing of intact rock material.
- Estimates of RQD from recovered cores and borehole televiewers.
- Spacing and persistence of discontinuities from recovered cores and borehole televiewers.
- Conditions of the discontinuities from recovered cores that may include dip angle, Jr, Ja, the joint roughness coefficient (JRC) or the joint wall compressive strength (JCS). The values of Jr and Ja may be determined following the rating system originally developed from Barton's Q tunneling index (Reference 02-3). JRC and JCS may be estimated in the

field following Barton and Choubey (Reference 02-4) and International Society for Rock Mechanics (ISRM) (Reference 02-5) for use with an appropriate shear strength criterion for discontinuities (e.g., Barton-Bandis criterion).

- Measurements of joint dip, dip direction and aperture based on the borehole televiewer.
- Groundwater conditions based on the wells and packer water-pressure tests.

The RMR will then be estimated using ratings based on the intact rock strength from recovered cores, the RQD from recovered cores and the televiewers, the spacing of discontinuities from recovered cores and the televiewers, the condition of the discontinuities from recovered cores and the televiewers, and a possible adjustment for the joint orientation relative to the structure or slope in accordance with EM 1110-1-2908 (Reference 02-6). The groundwater rating will be determined using the wells and packer water-pressure tests as further described in the GEH response to NRC Question 02.05.04-07.

The GSI can be estimated as described in Section 3.1.3 of NEDO-33914. This includes qualitative estimates of GSI charts by a qualified and experienced geologist or engineering geologist, and/or semi-quantitative estimates based on the measured RQD and the joint condition. Alternate methods for determining the joint condition are provided that include estimates of the joint roughness number (Jr) and joint alteration number (Ja) from Barton's Q tunneling index (Reference 02-3).

Depending on the rock mass strength and characteristics, in-situ tests such as rock pressuremeters (dilatometers) (Reference 02-7) and borehole (Goodman) jacks (Reference 02-8) may be used to estimate the rock mass deformation modulus during the subsurface investigation.

As shown in Figure 1-4 of NEDO-33914, the rock and soil media surrounding the Reactor Building (RB) shaft may not be accessible as the excavation progresses. Excavation support (e.g., slurry shoring wall or similar) will likely be needed for the RB shaft excavation that would make the soil inaccessible from the excavation. Depending on the subsurface conditions at a site, the rock excavation may be accessible for mapping beneath the soil excavation support. As described in Section 3.2.1 of NEDO-33914, mapping the walls and floors of the rock excavation during and at the completion of the rock excavation are intended to be the primary mechanism of confirming the assumptions made about rock media surrounding the RB shaft. Specifically, the orientation, joint characteristics, aperture, spacing, and persistence of the fractures used to characterize the rock mass could be confirmed. However, rock reinforcement, shotcrete, or waterproofing may limit access for mapping the rock excavation at some sites.

The excavated soils may be inspected to confirm the conditions and materials are consistent with the results from the Site investigation Program. If the foundation level of the RB shaft is founded in soil, in-situ tests to confirm the foundation conditions may be completed. As stated in Section 3.2.1 of NEDO-33914, the selected tests should confirm the key site parameters used to estimate the allowable static bearing capacity and maximum allowable dynamic bearing capacity. The selected test methods should be consistent with the test used to select the design values from the Site Investigation Program performed following the guidelines in Section 3.1 of NEDO-33914, RG 1.132 (Reference 02-1), and RG 1.138 (Reference 02-2).

When the rock excavation is accessible, additional in-situ testing may also be completed for rock and rock mass properties such as shear strength and deformation modulus. These tests may include larger-scale plate loading of the in-situ rock mass in the excavation (e.g., References 02-9 and 02-10) or in-situ direct shear tests on discontinuities exposed by the excavation (e.g., References 02-11 and 02-12). Overcoring with a borehole deformation gauge in the rock excavation (e.g., Reference 02-13) may also be implemented to confirm the in-situ stress field at some sites.

As stated in Table 3-1 of NEDO-33914, the televiewer locations are intended to characterize the rock mass and substitute for limitations on inspection during construction.

The eight geotechnical boring/borehole televiewer locations are intended to collect a substantial amount of data on the perimeter of the potential RB shaft excavation (Figure 3-1 of NEDO-33914). This information would include the recovered soil samples and rock cores at the geotechnical boring in the center of the RB shaft and the recovered soil samples, recovered rock cores and borehole televiewers at eight perimeter RB shaft locations to collect additional data on soil and rock properties. Geophysical surveys would also be completed in three of the geotechnical boring locations. The additional data are intended to provide substantial information on the soil and rock properties prior to excavating and sufficient information to compensate for limited inspections of the soil and rock mass when access is limited by the excavation methods.

References

02-1. RG 1.132, "Site Characterization Investigations for Nuclear Power Plants," Revision 2, October 2003.

02-2. RG 1.138, "Laboratory Investigations of Soil and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, December 2014.

02-3. Hoek, E., Carter, T.G., and Diederichs, M.S., "Quantification of the Geological Strength Index Chart," 47th U.S. Rock Mechanics/Geomechanics Symposium, San Francisco, California, June 23–26, 2013.

02-4. Barton, N., and Choubey, V., "The Shear Strength of Rock Joints in Theory and Practice, Rock Mechanics," No. 10 (1-2), pp 1-54, 1977.

02-5. International Society for Rock Mechanics (ISRM) Commission on Standardization of Laboratory and Field Tests, "Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses," International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, No. 15, pp 319-368, 1978.

02-6. USACE Engineer Manual EN 1110 1 2908, "Rock Foundations," U.S. Army Corps of Engineers, Washington, DC, November 1994.

02-7. American Society for Testing and Materials ASTM D8359 "Standard Test Method for Determining the In Situ Rock Deformation Modulus and Other Associated Rock Properties Using a Flexible Volumetric Dilatometer," 2021.

02-8. American Society for Testing and Materials ASTM D4971 “Standard Test Method for Determining In Situ Rock Modulus of Deformation of Rock Using Diametrically Loaded 76-mm (3-in) Borehole Jack,” 2016.

02-9. American Society for Testing and Materials ASTM D4506 “Standard Test Method for Determining In Situ Modulus of Deformation of Rock Mass Using Radial Jacking Test,” 2013.

02-10. American Society for Testing and Materials ASTM D4729 “Standard Test Method for In Situ Stress and Modulus of Deformation Using the Flat Jack Method,” 2019.

02-11. American Society for Testing and Materials ASTM D4554 “Standard Test Method for In Situ Determination of Direct Shear Strength of Rock Discontinuities,” 2012.

02-12. U.S. Army Corps of Engineers, “Suggested Method for In Situ Determination of Direct Shear Strength (ISRM),” RTH 321-80, Waterways Experiment Station, Vicksburg, MS, 1980.

02-13. American Society for Testing and Materials ASTM D4623 “Standard Test Method for Determination of In Situ Stress in Rock Mass by Overcoring Method—Three Component Borehole Deformation Gauge,” 2016.

Proposed Changes to NEDO-33914 Revision 0

Table 3-1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to indicate pressuremeter and/or borehole jack tests for making in-situ estimates of elastic moduli for the test purpose of geotechnical borings.

Sections 3.1.3 and 3.2.1 of NEDO-33914 are being revised as shown in the Enclosure 2 markups to identify the investigation locations and methods intended to characterize the rock and rock mass parameters and identify potential comparison to appropriate in-situ tests. Additional information is provided on the use of geotechnical and televiewer to provide additional data and characterize potentially in accessible portions of the excavation, and potential in-situ tests to verify select parameters in the excavation are identified.

Sections 3.2.1 and 5.2.1.2 of NEDO-33914 are being revised as shown in the Enclosure 2 markups to identify potential comparison to appropriate in-situ tests.

02.05.04-03 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Sections: TR NEDO-33914 Sections 3.2, 3.3, 3.4, and 4.1

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Revision 2, describes methods acceptable to the NRC staff for conducting field investigations to acquire the geological and engineering characteristics of the site and provides recommendations for developing site-specific guidance for conducting subsurface investigations.

RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, describes laboratory investigations and testing practices for determining soil and rock properties and characteristics needed for engineering analysis and design of foundations and earthworks for nuclear power plants.

Issue:

Section 4.1, Foundation Interface Analysis Model, states that a numerical model of the interfaces would be developed that examines the response of the BWRX-300 and its surrounding media due to alterations of in-situ subgrade conditions. The responses would be monitored, both through the FIA model response and field measurements. The numerical FIA model will also be calibrated using the field measurements to predict future response of the structure. It is not clear from the discussions in Sections 3.2, 3.3, and 3.4 how the predicted interface behavior would be compared against physical observations from the monitoring programs. Sections 3.2, 3.3, and 3.4 do not discuss any plan or program to monitor the shear and normal displacements along an interface, as shown in Figure 4-1.

Request:

The staff requests GEH to identify the plan(s) or program(s) to monitor the response of the BWRX-300 and its surrounding media and comparing them with predictions using the FIA model for

calibrating the numerical FIA model. Additionally, this process should verify that the structural and site responses are within the design bounds. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-03

The plan(s) and program(s) for monitoring the response of the BWRX-300 and its surrounding media shall be developed for each site based on the specific subsurface conditions. Licensing Topical Report (LTR) NEDO-33914 provides the general design approach and appropriate guidance to develop these plans and programs rather than a comprehensive plan or program for all sites.

The guidance for developing a suitable instrumentation and monitoring plan is provided in Figure 3-3 of NEDO-33914 with the required implementation periods for different monitoring instruments. Different responses shall be monitored at different stages of the project with a combination of permanent and temporary instrumentation. Water levels and pore pressures in soil and rock are monitored from the site characterization through operation. For the Reactor Building (RB) shaft, heave at the base of the excavation is temporarily monitored during excavation; wall displacements are temporarily monitored during excavation and construction; and, earth pressures are temporarily monitored through excavation, construction ,and loading. Settlement of the RB shaft and other structures and tilt of the RB shaft is permanently monitored through construction, loading, and operation of the BWRX-300.

As noted in Section 4.2 of NEDO-33914, the parameters defining the soil and rock constitutive models are developed based on data obtained from the field and laboratory testing programs and calibrated based on data collected from the field instrumentation program.

As described in Section 4.3.4 of NEDO-33914, temporary monitoring of heave, wall displacements, and earth pressures during the different stages of the project will allow for re-evaluation of the non-linear Foundation Interface Analysis (FIA) model parameters to better match the observed changes in stresses and displacement from different loading. Similar to Section 4.2 of NEDO-33914, these changes may include a revision to the constitutive models for soil and rock based on the uncertainties in the existing subsurface data, additional subsurface data collected at the site, and the observed response to the different loading conditions. The predicted and observed responses may include displacements at the base (heave) or along the walls of the RB shaft excavation, settlement or displacements adjacent to the RB shaft, and changes in earth pressures along the RB shaft. The revised parameters may then be used for prediction and comparison during the next project stage and subsequently revised, as needed, based on the observed responses.

Changes to the soil and rock constitutive models will impact the response of the interface models. The need for direct monitoring of interfaces will be a site-specific determination depending on their importance and the specific subgrade conditions.

Conservative soil and rock properties are developed following the guidelines in Section 5.2.1 of NEDO-33914 to serve as input for the design 1-g Soil-Structure Interaction (SSI) analyses described in Section 5.1 of NEDO-33914. Static earth pressure profiles obtained from the design

1-g SSI analyses are compared to the results of the non-linear FIA to confirm the design earth pressures from the design 1-g SSI analyses include adequate margin for uncertainties in subgrade properties surrounding the RB structure.

As described in Section 5.1.3 of NEDO-33914, a best estimate soil and rock pressure profile on the RB shaft is developed as an envelope of all maximum earth pressure values calculated by the non-linear FIA for all analyzed post-construction stages and scenarios. This pressure profile is compared to the pressure profile developed from the results of the design 1-g SSI analyses to confirm the linear elastic model provides adequate margins for the structural design. Soil and rock design pressure margins are calculated based upon the minimum values and the distribution of the ratio between the design soil and rock pressures obtained from the design 1-g SSI analyses and the best estimate pressures obtained from the non-linear FIA. If the values of the calculated soil and rock pressure margins are below the values deemed adequate to address the uncertainties and variations of subgrade properties, the rock mass weight or the equivalent linear soil and rock stiffness properties used for the design 1-g analyses are adjusted.

Probabilistic analyses may also be performed following the guidelines in Section 5.1.4 of NEDO-33914 to demonstrate the adequacy of the design by showing that the probability for the earth pressures exceeding the design earth pressure loads is low.

Proposed Changes to NEDO-33914 Revision 0

Section 3.4 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to indicate direct monitoring of interfaces will be a site-specific determination depending on the importance of the interface and the specific subgrade conditions.

02.05.04-04 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

In Section 5.1.2, Soil-Structure Interaction Modeling Assumptions, the rock mass is assumed to be continuous, and the presence of cavities, fracture zones, joints, bedding planes, discontinuities, and other weak zones is neglected. It is not clear from the discussion whether their effects on the rock mass properties (e.g., rock mass modulus, strength) would be incorporated through equivalent rock mass properties so that the calculated loads on the RB walls are realistic. It is also not clear whether an isotropic assumption of the equivalent material properties would be made. Rock fractures have specific orientations in the 3D space and make the rock mass properties anisotropic.

Request:

The staff requests GEH to provide a discussion in the TR how the effects of rock fractures etc. would be incorporated in the SSI modeling.

GEH Response to NRC Question 02.05.04-04

As described in Licensing Topical Report (LTR) NEDO-33914, Section 5.1.2, the design Soil-Structure Interaction (SSI) analyses used for calculating static and seismic demands for design of Reactor Building (RB) structures are performed based on the principles of continuum mechanics. The material properties, including those of the rock, are assumed to be isotropic and linear elastic. Therefore, possible fracture zones, joints, bedding planes, discontinuities and cavities in the rock are not explicitly included in the design SSI analyses that are performed using linear elastic models described in Section 5.0 of NEDO-33914. Assumption (1) in Section 5.1.2 of NEDO-33914 is being revised to clarify that the properties of subgrade materials, including the rock, are assumed to be isotropic and linear elastic.

To ensure the RB design bounds the effects of rock fractures, cavities and other types of rock features that do not conform to the modeling assumptions adopted for the design SSI analyses, deterministic and/or probabilistic evaluations are performed following the guidelines provided in Sections 5.1.3 and 5.1.4 of NEDO-33914, respectively. These evaluations compare the results of 1-g SSI analyses that provide the static stress demands due to earth's gravity for the design of the RB structure, with the results of non-linear Foundation Interface Analysis (FIA) and/or limit equilibrium analyses performed using models that include the effects of fracture zones, joints, bedding planes, discontinuities or cavities in the rock. For sites characterized with a high non-linear behavior of the subgrade materials and high seismicity, sensitivity seismic analyses may be performed on non-linear SSI models following the approaches described in Section 5.3.11 of NEDO-33914 to evaluate the effects of rock fractures, cavities and other types of rock discontinuities on the seismic response and design of the BWRX-300 RB.

The design 1-g SSI analyses described in Section 5.1 of NEDO-33914 use linearized properties of soil and rock materials to provide upper bound estimates of stress demands for the design of the deeply embedded RB structure. Lower bound elastic modulus values are paired with upper bound unit weight and Poisson's ratios to emphasize the subgrade deformations and the soil and rock pressures on the below-grade RB walls. Lower bound elastic modulus values are obtained from the empirical equations presented in Section 5.2.1 of NEDO-33914 using conservative estimates of the intact rock modulus and rock mass classification parameters. The rock mass classification parameters, such as the Geological Stress Index (GSI) and Rock Mass Rating (RMR), are based on the results from site investigation and subsurface material testing programs in Section 3.1 of NEDO-33914. Upper bound Poisson's ratios and unit weights are used to emphasize soil and rock pressures on the RB shaft.

Unit weights are assigned to the rock masses that require lateral support when excavated to account for the potential rock pressure applied to the RB shaft. Very small (close to zero) unit weight values can be assigned to the self-supporting rock masses. Please refer to the GEH response to NRC Question 02.05.04-06.

For site-specific conditions where the soil and rock pressures from the design 1-g SSI analyses do not bound the results of non-linear FIA and/or limit equilibrium analyses, adjusted rock elastic modulus and Poisson's ratios are used as input for the design SSI analyses to obtain conservative design demands that include adequate margins to envelope the effects of possible anisotropic

behavior of rock masses. The adequacy of the design margins is evaluated based on the levels of uncertainty present in the determination and modeling of the site-specific subgrade conditions.

Poisson's ratio, the unit weight, and the elastic modulus can be adjusted based on the soil and rock pressures from non-linear FIA models that include the potential rock discontinuities, as described in NEDO-33914, Sections 4.2.2 and 4.3.1.2, and/or force limit equilibrium analyses, as illustrated in Figure 5-1 of NEDO-33914. In these cases, lateral soil and rock pressures are calculated using estimates obtained from the non-linear FIA and/or limit equilibrium analyses and used in NEDO-33914 Equation (5-14) to determine modified Poisson's ratio values. These modified Poisson's ratios and unit weight properties are assigned to the rock mass to ensure the design 1-g SSI analyses results adequately bound the results from the non-linear FIA and/or limit equilibrium analyses. Additionally, the intact rock modulus values and/or rock mass classification parameters can also be modified based on the range of potential parameters established in Section 3.1 of NEDO-33914.

The Generalized Hoek-Brown (GHB) rock model described in Section 4.2.2 of NEDO-33914 is typically used to simulate jointed rock masses where the response of the rock stiffness is nearly constant over a range of stresses, but the shear strength is variable due to the presence of discontinuities and weak zones. Even though the GHB model does not explicitly simulate discrete joints and fractures with anisotropic behavior, this behavior is considered through the GSI that is estimated based on joint structure and surface quality. This equivalent approach is appropriate to simulate jointed rock masses.

The seismic design SSI analyses are performed using a minimum of three profiles of subgrade dynamic properties to account for the variability and uncertainties in the subgrade material properties. The strain compatible dynamic properties that are developed for the subgrade materials as described in Section 5.2.4 of NEDO-33914 address the effects of primary non-linearity of soil materials. The dynamic stiffness and unit weight properties of the rock masses may be adjusted using an approach similar to ones used for adjusting the static rock properties to account for possible amplifications of dynamic earth pressures on the RB shaft due to rock discontinuities, fracture and weak zones.

Proposed Changes to NEDO-33914 Revision 0

Section 5.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to note that:

- the elastic modulus and Poisson's ratios may be adjusted based on the results of the FIA to obtain bounding soil and rock pressures from the design 1-g SSI analyses performed on models with isotropic linear-elastic properties to envelope the effects of possible anisotropic behavior of rock masses,
- the adequacy of the design margins to address uncertainties in determination and modeling of subgrade conditions is addressed on a site-specific basis,

- non-linear FIA and limit equilibrium models may also be used to evaluate potential loads from rock blocks and wedges and swelling or squeezing soil and rock units and adjust the equivalent liner subgrade properties for use as input for the design 1-g SSI analyses, and
- the seismic design SSI analyses may use dynamic stiffness and unit weight properties of the rock masses that are adjusted using an approach similar to the ones used for adjusting the static rock properties.

02.05.04-05 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

Section 5.1.2, Soil-Structure Instruction Modeling Assumptions, states that "Strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness will frequently be self-supporting even if some reinforcement is required to ensure a safe excavation." It is not clear what is meant by disadvantageous fracture zones, joints, bedding planes, discontinuities, and other zones of weakness, and how they will be identified at a site.

It is further stated that "Joints and other weak planes may create isolated blocks that are unstable; however, these blocks are not typically large relative to the area of the structure and would be unlikely to produce significant loads on the exterior of the structure compared to other loads (e.g., hydrostatic). These blocks would also not be able to create a cascading failure once the structure is in place." It is not clear what are the basis for the assumption that unstable blocks would be isolated. It is also not clear why the unstable blocks would not produce significant loads on the RB structure. The unstable blocks could impose concentrated load(s) with significantly higher magnitude than the hydrostatic load on the RB walls (e.g., the scenario shown in Figure 5-1). It is also not clear from the discussion how the design of the RB structure would account for such large rock mass failure.

Request:

The staff requests GEH to provide an approach to identify the disadvantageous fracture zones, joints, bedding planes, discontinuities, and other zones of weakness at a site. The staff also requests GEH to provide rationale why the unstable blocks would not produce significant loads on the RB structure and explain how the design of the RB structure would account for such load. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-05

Disadvantageous fracture zones, joints, bedding planes, discontinuities, and other zones of weakness at a site will be identified by the subsurface investigation described in Section 3.1 of Licensing Topical Report (LTR) NEDO-33914. Specifically, disadvantageous features that may result in pressure from the rock are intended to be identified by the geotechnical borings and borehole televiewer locations as described in the GEH response to NRC Question 02.05.04-02. Based on this focused subsurface data at the Reactor Building (RB) shaft, potentially disadvantageous weak zones that could invalidate the self-supporting rock assumption may be identified. Subsequent estimates of the strength of the intact rock and along these potential weak zones through laboratory and/or in-situ testing may be used to estimate the stability and/or pressures for these rock masses or potential unstable blocks and wedges by using either a deterministic approach, described in Section 5.1.3 of NEDO-33914 or the probabilistic approach, described in Section 5.1.4 of NEDO-33914.

The text describing the smaller loads on the RB structure from isolated blocks will be removed from NEDO-33914. The text does not aid in defining the modeling assumption for the one-step design approach regarding self-supporting rock as discussed in the GEH response to NRC Question 02.05.04-06.

Evaluations of the potential rock pressures are performed for rock sites to either confirm self-supporting rock is present or to estimate the rock pressures after degradation of the reinforcement. Non-linear Foundation Interface Analysis (FIA) and/or limit equilibrium analyses are performed to estimate the likely pressures from the soil, rock, and water on the RB shaft. The GEH response to NRC Question 02.05.04-06 provides a description of how loads from the rock or rock mass are considered in the design 1-g Soil-Structure Interaction (SSI) analyses of the RB shaft through comparisons to the non-linear FIA and/or limit equilibrium analyses.

The validation of soil and rock pressure loads may consider the subgrade improvements like consolidation grouting, rock reinforcement, and soil support made during the construction. However, these improvements are typically considered only as initial ground support that is separate from the permanent ground support system because these types of reinforcements and any surface protection will be inaccessible for monitoring and repair after the construction.

The rock pressure on the RB shaft wall may be considered uniform with contact grouting to avoid stress concentration or point load associated with the block or wedge that is reinforced to stabilize the rock excavation. The evaluation of these rock pressures assumes that the excavation has

reached stability with initial rock support and that the liner will accept one hundred (100) percent of the initial rock support as it relaxes over the lifetime of the structure. The initial rock support loads should be conservative since rock loads in stressed rock masses are typically not following (e.g., they typically reduce with displacement due to arching). The notable exception would be due to the presence of hydrostatic loads and swelling or squeezing rock displacements that are time dependent and can continue to apply a large load with continued displacement.

When pressures from swelling or squeezing rock displacements create the potential for loads on the RB shaft, the RB shaft can be designed to take the full pressure, a compressible material can be placed between the RB shaft and rock to reduce the pressures, or a scheduled construction delay can be added to allow the deformation to occur and reduce the pressures. If the degradation of initial support for large rock blocks potentially creates unacceptable high pressures, other options can include overexcavating and backfilling the rock block to reduce the potential pressures, the use of degradation resistance rock reinforcement to permanently support the rock block, or changes in the BWRX-300 location to improve the relative position of the rock block and reduce the potential pressures on the RB shaft.

Proposed Changes to NEDO-33914 Revision 0

Section 5.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to remove the text stating isolated unstable blocks do not produce significant loads. The text is modified to indicate rock reinforcement is typically considered as initial ground support.

Section 5.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to provide potential options to address large rock pressures when swelling or squeezing rock displacements or unstable rock blocks are identified.

02.05.04-06 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

Section 5.1.2, Soil-Structure Instruction Modeling Assumptions, assumes that the rock is self-supporting. It is not clear whether the BWRX-300 reactor system cannot be installed in a rock mass that is not self-supporting, e.g., rock mass with poor rock quality.

Request:

The staff requests GEH to clarify whether a site requiring significant permanent support system(s) to keep the surrounding media stable would be unsuitable for siting a BWRX-300 reactor. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-06

The BWRX-300 reactor can be designed to be deployed at soil sites and sites having rock masses that require support during the excavation and construction of the deeply embedded Reactor Building (RB) shaft. The methods noted in Section 3.1.3 of Licensing Topical Report (LTR) NEDO-33914 can be deployed to support the excavation of rock masses with weak zones. The

design considers the loads from rock that is not self-supporting by using equivalent linear-elastic rock properties that provide upper bound soil and rock pressures and stress demands for the design of the RB structure.

Upper bound estimates of the unit weights are assigned to rock masses that require support during excavation and construction. Strong rock, where stability during excavation is not compromised by fracture zones, joints, bedding planes, discontinuities and other zones of weakness, is considered self-supporting. Very small (close to zero) unit weights can be assigned to these self-supporting rock masses in the models for the design 1-g SSI analyses.

As discussed in the GEH response to NRC Question 02.05.04-04, earth pressure loads from the design 1-g SSI analyses are confirmed as adequate upper bound estimates by their comparison to the soil and rock pressures obtained from the non-linear FIA and/or limit equilibrium analyses. Deterministic and/or probabilistic evaluations are performed following the guidelines provided in Sections 5.1.3 and 5.1.4 of NEDO-33914, respectively, to ensure the design includes adequate margins to account for the uncertainties related to the earth pressures applied on the RB structure from rock that is not self-supporting. The adequacy of the margins is determined on a site-specific basis as noted in the GEH response to NRC Question 02.05.04-04.

For sites characterized with a high non-linear behavior of the subgrade materials and high seismicity, non-linear seismic Soil-Structure Interaction (SSI) analyses may be performed, following the approaches described in Section 5.3.11 of NEDO-33914, to evaluate the effects of rock masses with poor quality and weak zones on the design of the RB structure.

Proposed Changes to NEDO-33914 Revision 0

Assumption (4) in Section 5.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to clarify that the design analyses can only neglect the rock pressures due to the weight of rock masses that are not required to be laterally supported during the excavation and construction.

02.05.04-07 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 4.3.3

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Issue:

Section 4.3.3, Fluid-Soil Interaction, states that the 3D model of BWRX-300 may have hydraulic interface(s) to simulate the effects of pore water during excavation, construction, loading, and operation phases of the reactor. In rock, flow through the rock fracture network can be the dominant flow mechanism. It is not clear what approaches would be taken to deal with fracture flow if it is present.

Request:

The staff requests GEH to clarify the approach to account for fracture flow. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-07

Explicit simulation of fractured flow is not anticipated for the model outlined in Section 4.3.3 of Licensing Topical Report (LTR) NEDO-33914. The focus of the non-linear Foundation Interface Analysis (FIA) would be to model the potential groundwater flow due to groundwater level changes to represent high and low water tables or dewatering activities. Therefore, modeling the rock as a continuum is considered sufficient to model local changes in the groundwater levels for most sites. The fractured flow of groundwater in rock will likely be affected or reduced by grouting and/or dewatering near the excavation for the Reactor Building (RB) shaft for construction activities as noted in Section 3.1.3 of NEDO-33914.

Characterization for rock mass properties can address the influence of the groundwater conditions and provide input parameters for subsequent analyses. The Rock Mass Rating (RMR) system uses a groundwater parameter and rating value that can be used to account for the groundwater in the fractured rock mass without modeling the fractured flow. The RMR groundwater rating can be based on the water levels in piezometers and the results of water-pressure tests in rock units as indicated in Table 3-1 of NEDO-33914. A range of ratings may be considered to address uncertainty, but flowing conditions should be considered in rock units below the water table where water-pressure tests indicate flow in discontinuities. The Geological Strength Index (GSI) incorporates the potential deterioration of shear strength along rock discontinuities by recommending a shift to lower values in rocks with fair to very poor discontinuity surface conditions. Water pressure does not modify the GSI value; however, it is included by effective stress analysis using an appropriate constitutive model (e.g., Generalized Hoek-Brown, Mohr Coulomb).

Proposed Changes to NEDO-33914 Revision 0

Section 3.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to include characterization and consideration of the groundwater conditions in evaluating rock mass characterization.

02.05.04-08 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 3.1.1

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Issue:

The TR does not state whether there would be a program to measure the in-situ stress field at the site. In Section 3.1.1, Site Investigation Program, the maximum required drilling depth is set at 120 m because the expected change in stresses due to excavation of the shaft would be less than 10% from the original in-situ stress field. It is not clear how this can be set without knowing the in-situ stress field. The stress distribution around the RB shaft could be quite different if horizontal stresses are larger than the vertical stress affecting the loads on the RB shaft walls.

Request:

The staff requests GEH to clarify whether there will be process to measure the in-situ state of stresses at the site and incorporate the stress field in the analyses. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-08

Consistent with Appendix D of RG 1.132 (Reference 08-1), the maximum required drilling depth for engineering purposes of 120 m is based on a change in the vertical stress.

The Licensing Topical Report (LTR) NEDO-33914 is being modified to clarify that the dmax value is based on the vertical stress. This definition of dmax is consistent with RG 1.132 (Reference 08-1) with estimates of vertical stress from parameters like the unit weight of soil and rock, and the water table depth.

The initial stress field at a site may be important for modeling the response of the site to the excavation and construction of the Reactor Building (RB) shaft, in particular when regional and/or local information indicates the rock mass may have residual horizontal stresses from tectonic activity, residual strains, or topographic conditions that are larger than the vertical stresses (Reference 08-2). The potential impact of the horizontal stresses and time-dependent deformation of the rock would be considered in the design and construction of the RB shaft. As shown in Table 3-1 of NEDO-33914, these measurements of the in-situ stress are performed as part of the geotechnical borings and borehole televiewer tests. Overcoring and hydraulic fracturing are identified as potential methods for estimating the in-situ stress; however, other methods including the identification of the principal stress directions and relative magnitudes from breakouts recorded by borehole televiewer may be used (e.g., Reference 08-3). Other in-situ stress tests may be performed following the guidance of RG 1.132 (Reference 08-1) to determine stress conditions in soil and rock. The appropriate tests for measurement of in-situ stress will be selected based on the likelihood for residual horizontal stresses and the type of subsurface materials at the site. All tests for measuring in-situ stress are not identified in NEDO-33914 since the appropriate tests will be specific to each site.

References

08-1. RG 1.132, "Site Characterization Investigations for Nuclear Power Plants," Revision 2, October 2003.

08-2. U.S. Nuclear Regulatory Commission, "Field Investigations for Foundations of Nuclear Power Facilities," NUREG/CR-5738, ADAMS Accession No. ML003726925, November 1999.

08-3. Science Applications International Corporation (SAIC), "Final Report – In Situ Stress Measurements in the NPR Hole," Report Submitted to Westinghouse Savannah River Company, ADAMS Accession No. ML013190312, July 1992.

Proposed Changes to NEDO-33914 Revision 0

Section 3.1.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to define dmax as based on the vertical stress and as the depth for engineering purposes. Additionally, the need for deeper borings for soil-structure interaction studies is indicated.

Table 3-1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to correct "in-site stress" to "in-situ stress" for the test purpose of geotechnical borings and borehole televiewer.

Section 4.3.4.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to say the initial stress conditions include the influence of groundwater and residual horizontal stresses.

02.05.04-09 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.4

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

Section 5.1.4, Probabilistic Earth Pressure Analyses, presents an approach as an example to account for the uncertainties in the estimated value of the load on the RB shaft walls in a soil medium. It is not clear whether similar approaches would be used to account for uncertainties in at least important parameters significant to the reactor design; for example, the estimated rock mass modulus estimated using various empirical equations from different measured and inferred parameters (indirect estimation).

Request:

The staff requests GEH to clarify whether there will be a plan to account for the uncertainties in other site-related parameters with potential to significantly affect the reactor design. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-09

Section 5.1.4 of Licensing Topical Report (LTR) NEDO-33914 presents a general probabilistic approach for ensuring that the Reactor Building (RB) structural design based on demands obtained from the one-step approach analyses of linear elastic models bounds the uncertainties related to:

- Site parameter uncertainties related to natural randomness, measurement errors and other uncertainties related to the methods or empirical relationships used for development of mechanical properties of subgrade materials; and
- Modeling uncertainties related to the simplifying modeling assumptions discussed in Section 5.1.2 of NEDO-33914.

The approach ensures that the probability for the earth pressures on the RB below-grade shaft to exceed the design earth pressure loads calculated by the 1-g Soil-Structure Interaction (SSI) analysis is low. This probabilistic approach is intended to be used for sites where probability calculations are required to demonstrate that the available design margins are adequate.

Discrete probability distributions are calculated using different approaches or empirical relationships to account for the uncertainties in the methods used for the development of the site parameters, such as the uncertainties in the estimation of rock mass modulus that can be obtained using various empirical equations using different measured and inferred parameters. These discrete probability distributions are combined following the recommendations in Section 5.1.4.4 of NEDO-33914. For example, a set of discrete probability distributions can be calculated using various empirical equations for estimating the rock mass modulus considering the probability distribution of different measured and/or inferred parameters. These discrete probability distributions are combined to obtain the probability distribution representing the uncertainties related to the calculations of rock mass modulus.

The earth pressure from the surrounding subgrade is the most important site-related load for the structural integrity of the deeply embedded RB. Therefore, the earth pressure load probability density functions will account for uncertainties in the most important site subgrade parameters that can affect the design of the RB structure, including the uncertainties related to the calculations of rock mass modulus if applicable.

Displacement results obtained from the non-linear Foundation Interface Analysis (FIA) described in NEDO-33914 Section 4.0 may also be used for specific site conditions where the consideration of earth pressures alone cannot completely address all uncertainties related to the subgrade stiffness parameters. For these sites, the effects of uncertainties in subgrade stiffness parameters on the structural integrity of the RB can be evaluated by comparing the displacement results of the FIA and the design 1-g SSI analysis at the interfaces of the RB structure with the surrounding soil and rock.

Alternatively, the FIA calculated displacements can be applied to the RB structural design analysis model as boundary conditions at the soil-structure interfaces to calculate stress responses of selected major RB structural members that are in line with the FIA calculated subgrade response. These FIA compatible structural stress results can then be compared to the corresponding results of the design 1-g SSI analysis to demonstrate that the design adequately address the uncertainties

related to the determination of subgrade stiffness parameters. For sites where probability calculations may be required to demonstrate that the available design margins are adequate, probability density functions of SSI structural deformations can also be developed from results of probabilistic analyses performed on simplified FIA models following the approaches described in Section 5.1.4 of NEDO-33914. These deformation probability functions can be then used as input for a probabilistic structural analysis to calculate the probability of the structural stresses responses to exceed the stress demands used for the design of the RB structure.

Proposed Changes to NEDO-33914 Revision 0

The first three paragraphs of Section 5.1.4 of NEDO-33914 are being revised as shown in the Enclosure 2 mark-up to explain that the probabilistic earth pressure evaluations also consider uncertainties in the methods or empirical relationships used for development of site-related parameters following an approach similar to the approach used to account for modelling uncertainties.

Table 5-1 of NEDO-33914 is being revised as shown in the Enclosure 2 mark-up to indicate that the models for probabilistic analyses also consider the natural and measurement uncertainties in rock mass properties like intact rock elastic modulus and strength parameters.

Paragraphs are added at the end of Sections 5.1.3 and 5.1.4 of NEDO-33914 as shown in the Enclosure 2 mark-up to explain that the approaches for validation and evaluation of uncertainties of earth pressure loads can also be used to evaluate effects of uncertainties in subgrade stiffness parameters on the RB structural design.

information gaps between the existing available information and specific needs for the siting of the BWRX-300.

3.1.1 Site Investigation Program

Figure 3-1 represents a preliminary layout of the BWRX-300 footprint and facilities with the deeply embedded RB being the only SC-I structure in the BWRX-300 plant. It is common practice to perform borings and tests below the footprint of the SC-I facilities and to deeper depths than the basemat (RG 1.132, Reference 8.64). The excavation approach minimizes the use of engineered backfill materials as well as the deployment depth of the BWRX-300 RB and requires a subsurface investigation that covers areas beyond its foundation perimeter.

The diameter of the RB SC-I footprint is relatively small when compared to footprints of typical conventional nuclear plants. The characterization of a small portion of the subsurface environment would be insufficient to adequately characterize the variations and uncertainties in the site subsurface conditions and provide inputs for the Approach 3 probabilistic SRA described in Section 5.2.2. Tests, such as seismic refraction or reflection studies that are useful to map bedrock or detect potential voids become meaningful and possible only when covering greater areas. Measurements of shear-wave velocities (V_S) and compression-wave velocities (V_P) are not sufficient to characterize lateral variability if these are made just a few meters apart.

In order to address the specific requirements of the BWRX-300 RB design, the subsurface site investigations are performed following the guidelines of RG 1.132 (Reference 8.64) for SC-I type site investigations considering the combined footprint areas of the RB SC-I foundation and the adjacent TB, CB and RwB foundations. The extended area considered by the BWRX-300 subsurface site investigation ensures an adequate characterization of the subsurface conditions under the TB, RwB and CB foundations and resulting surcharge loads, which are important for the design of the deeply embedded RB structure and seismic design of RB SC-I SSCs.

Appendix D of RG 1.132 (Reference 8.64), Spacing and Depth of Subsurface Explorations for Safety-Related Foundations, specifies the need for at least one boring underneath each projected safety-related structure or 1 boring for each 900 m^2. The footprint of the main containment shaft and the above ground surrounding structures is about 1 Ha (10,000 m^2). This implies that at least 10 borings would be required for the site investigation. RG 1.132 (Reference 8.64) indicates that the boring depth for soil-structure interaction studies should go past "the maximum required depth for engineering purposes." If bedrock is encountered, then the boring should penetrate past zones of weakness that could affect foundation performance and extend at least 6 m into sound rock. For the BWRX-300, the maximum required depth for engineering purposes d_{max} is set at approximately 120 m, a depth that is the greater than the following:

a) The depth of the shaft plus twice the diameter of the shaft, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition, and

b) Twice the width of the plant's footprint, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition.

The extent and detail of the site investigation depend on the encountered subsurface conditions. Table 3-1 lists the expected types and amounts of tests that are required to properly characterize

site conditions. A boring and geophysical exploration layout is given on Figure 3-1. Table 3-2 lists the recommended borings and their purpose. A minimum of 21 boring locations is anticipated within the BWRX-300 site investigation program which exceeds the minimum of 10 borings based on recommendations in RG 1.132, Appendix D (Reference 8.64). The increase in the number of borings is to ensure adequate characterization of subsurface properties under and around the deeply embedded RB structure. Previously investigated sites may have information that cover a wide area. In such cases, only limited and targeted additional exploration points may be required.

Table 3-1: Site Investigation for the BWRX-300

Test Type		Test Purpose	Number of Tests [1]
1	Geotechnical borings	− Measure Standard Penetration (SPT) − Measure Cone Penetration Resistance − Sample soils and rock for visual classification and laboratory testing − Rock Quality Designation (RQD) − Perform pressuremeter and/or borehole jack tests on weak to moderately soft rock portions to have data parameter for estimation of elastic moduli − Measure in-~~-~~situ~~e~~ stress (overcoring, hydraulic fracturing)	− 3 borings at perimeter and center of containment down to 120 m − 2 borings at perimeter of containment down to a depth of 60 m − About 18 additional borings designed to cover the footprint of the main facilities, meet the regulatory guidance, and characterize the subsurface as a unit. (see Figure 4-1)
2	Wells	− Groundwater characterization (pump and slug tests, baseline groundwater quality) − Characterize groundwater flow direction and quantify hydraulic gradients	− 9 wells at the center and edge of containment to anticipated depth of 60 m − 4 wells down to a depth of 60 m covering the footprint of the facility
3	Geophysical boring	− Measure V_P and V_S with at least two methods: seismic downhole survey, crosshole, and/or and PS Log suspension survey.	− One boring down to 120 m at center − 4 borings at perimeter of containment down − 4 borings located a distance apart from RB to allow for wider cross sections and correlations to refraction or reflection surveys
4	Refraction Survey	− For sites in which a bedrock horizon is identified by the boring program, perform seismic refraction to obtain a three-dimensional mapping of the bedrock horizon and the thickness of weathered layers	− One grid of surveys covering the footprint extension of the facility
5	Seismic reflection survey	− Identify if voids, sinkholes, karst, or faults are present beneath the footprint of the facilities	− Three longitudinal and two to three transverse reflection sections
6	Borehole Televiewer (Optical/Acoustic)	− Observe rock surface directly, subsurface lithology and structural features such as fractures, fracture infillings, foliation, and bedding planes. − Packer water-pressure tests in rock − Measure in-~~-~~situ~~e~~ stress (overcoring, hydraulic fracturing)	− Relevant for rock conditions, over which boring recovery and RQD allow for an open borehole. − The proposed 8 televiewer locations will support a better characterization of the rock mass and as a substitute for potential inspection limitations due to the construction process.

[1] Number may be adjusted depending on encountered site conditions and site available information

Table 3-2: Anticipated Boring Program

Boring	Depth (1) (m)	SPT(2)/ CPT(3) Coring	VEL DH(1)	VEL PS LOG(2)	Well
B-01	120	✓	✓	✓	✓
B-02	120	✓	✓	✓	✓
B-03	120	✓	✓	✓	✓
B-04	60	✓			✓
B-05	60	✓			✓
B-06	60	✓			
B-07	30	✓			
B-08	60	✓			
B-09	80	✓	✓	✓	
B-10	80	✓	✓	✓	
B-11	60	✓			
B-12	60	✓			
B-13	80	✓			✓
B-14	80	✓	✓	✓	
B-15	60	✓			
B-16	80	✓			✓
B-17	60	✓			✓
B-18	80	✓	✓	✓	
B-19	60	✓			✓
B-20	60	✓			
B-21	100	✓			
TOTAL	~1600	~21	~7	~7	~9

Notes:
(1) Subject to change based on site conditions
(2) SPT: Standard Penetration Test
(3) CPT: Cone Penetrometer Test
(4) VEL DH: Downhole velocity (VEL) test
(5) VEL PS Log: PS Suspension log velocity (VEL) test

3.1.2 Laboratory Testing Program

A laboratory testing program is performed on soil and rock samples collected from the site investigation program in accordance with the regulatory guidance of RG 1.138 (Reference 8.65) to obtain data for the analysis and design of the BWRX-300 RB. The scope and extent of the BWRX-300 laboratory testing program address the specific requirements of deeply embedded BWRX-300 design that requires a reliable set of data from laboratory tests for developing geotechnical inputs characterizing the properties of each subgrade material present at the site. Testing to estimate parameters for appropriate soil and rock discontinuities should be included using the methods identified in RG 1.138 (Reference 8.65). Testing of artificial interfaces for the foundation materials or use of adjacent material properties and sensitivity analyses, as described in Section 4.3.1.1, Interfaces Between the Structures and Subgrade Media, can be considered.

A laboratory testing program is implemented that depends on the site-specific subsurface conditions, the specific analysis requirements, and the need for sufficient data to adequately characterize variations in subsurface material properties. A sufficient number of laboratory tests are performed to minimize the uncertainties in the design related to these geotechnical input parameters by providing reliable estimates for the statistical parameters (mean and standard deviation values) of the measured material properties. The systematic (bias) errors are minimized by a carefully executed equipment calibration and sample management programs. Estimates of measuring bias are developed based on comparisons of measurements of physical parameters obtained from different types of subsurface material property tests.

At a minimum, the laboratory tests of soil materials include:

- Index testing (classification, weight, plasticity, grain size)
- Strength testing (shear tests, triaxial tests)
- Deformability tests (triaxial tests, consolidation tests)
- Permeability
- Chemical testing (chlorides, sulfates, pH, Resistivity)
- Dynamic tests (Resonant Column Torsional Shear (RCTS), cyclic triaxial)

The minimum laboratory tests required to develop properties for rock materials include:

- Uniaxial Compressive (UC) strength,
- Triaxial compressive strength and elastic moduli,
- Direct shear tests,
- Petrography,
- Dynamic tests (sonic pulse wave velocity, Free-Free Resonant Column velocity tests)

Other tests, such as the expansion, creep, mineralogy, erodibility, durability, X-ray diffraction tests may be performed on an as-needed basis.

3.1.3 Characterization of Rock Mass Properties

The properties of rock are characterized based on the information collected from the site investigation and laboratory testing programs described in Sections 3.1.1 and 3.1.2. Rock joints, bedding planes, discontinuities fracture and other weak zones are evaluated to determine:

- the type of temporary excavation support and improvements required during construction;
- groundwater conditions and required seepage control measures; and
- possible effects on the rock pressure loads on the RB shaft.

The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones may affect methods used to excavate rock for construction of the shaft. Methods that are used to compensate for these weak zones include:

- over-excavation and backfilling;

- internal structural support;
- spot or pattern rock reinforcement (i.e., rock bolts or anchors); and
- surface treatments (i.e., mesh, straps, shotcrete).

Additionally, the existing groundwater conditions and the potential control of seepage through cavities, fracture zones, joints, bedding planes, and discontinuities is considered. Seepage control may include slurry walls, grouting prior to excavation, grouting during the excavation, freezing, drains, dewatering wells, sumps and other methods. The existing groundwater conditions and the impact from potential deterioration of shear strength or a reduction in rock mass quality is based on the results of piezometers and water-pressure tests described in the Site Investigation Program in Section 3.1.1.

Discontinuities and other zones of weakness within the rock mass may also control the stability of individual blocks or the rock mass when the orientation is disadvantageous and/or the spacing of discontinuities is sufficiently dense. The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. These discontinuities or weak zones may form a system of blocks or wedges where strength within the individual blocks is high, but strength along the weak zones between the blocks is highly anisotropic.

To adequately assess and consider weak zones in rock masses, RG 1.132 (Reference 8.64) and NUREG/CR-5738 (Reference 8.2) provide guidance on logging and characterizing rock materials. Frequently, optical and acoustic televiewers (OTV/ATV) are used in conjunction with oriented or classical rock coring methods to map the depths, spacing, orientations, aperture, and other characteristics of the discontinuities. The geotechnical borings, wells, and borehole televiewer locations identified in Table 3-1, Site Investigation for the BWRX-300, are intended to collect rock data, groundwater information, and samples for laboratory testing to classify the rock mass. The type of information and testing required for the rock mass will depend on the specific subgrade conditions as well as the rock mass classification selected for the site. Inclined borings may be used to properly characterize the orientation of near vertical discontinuities.

Empirical engineering and geo-mechanical rock mass classifications, such as the Rock Quality Designation (RQD) index, the Rock Tunneling Quality (Q) index, the 1976 and 1989 versions of the Rock Mass Rating (RMR) system, and the Geologic Strength Index (GSI), are used to quantitatively characterize the geologic and engineering parameters of rock masses (FHWA, 2009). These classifications often consider a variety of parameter ratings that are assigned based on the observations and measurements from characterized rock mass and may incorporate the proposed excavation techniques. Frequently, a range of parameter ratings are considered because a range of rock mass characteristics are encountered during subsurface characterization and multiple classifications systems may be considered to incorporate uncertainty in the parameter estimates.

Estimates of RQD may be made following NUREG/CR-5738 (Reference 8.2) on recovered rock cores and confirmed using OTV/ATV data or estimated from mapped or scanned surfaces based on the average number of discontinuities or volumetric joint count (Hoek et al. 2013, Reference 8.10).

RMR may be estimated following the parameters and ratings established by Bieniawski (1976, 1989, Reference 8.11). In order to use the RMR system, a rock mass is divided into different structural units defined by changes in rock type or major changes within a rock type, such as faults, fracture zones, or the spacing of discontinuities that may cause a change in the rock mass behavior. The RMR then considers semi-quantitative parameters for each structural region, which include the strength of the intact rock, RQD, the spacing of discontinuities, the condition of the discontinuities, the groundwater conditions, and the orientation of the discontinuities. Estimates of these parameters would be based on the data from the Site Investigation and Laboratory Testing programs. Even though GSI is now commonly used directly without an estimate based on RMR, RMR is retained because previous studies have indicated better estimates using RMR for the rock mass deformation modulus of moderate to strong rock masses (Galera et al., 2007, Reference 8.12). Comparisons can be made to in-situ measurements of rock modulus identified in Sections 3.1.1 and 5.2.1.2.

GSI may be estimated using qualitative charts relating the structure of the rock to the surface condition of joints for different types of rock masses (e.g., Hoek and Brown, 2018, Reference 8.13). Originally, the GSI system was developed for rock masses where block sliding and rotation was the primary means of failure without failure of the intact rock blocks, but has been extended to additional charts for other types of rock masses and geologic environments (Hoek and Brown, 2018, Reference 8.13). An appropriate GSI chart must be selected for the project site.

GSI may also be estimated semi-quantitatively for rock masses where block sliding, and rotation is the primary means of failure. This semi-quantitative method was developed for use when a qualified and experienced geologist or engineering geologist does not observe the rock mass and is recommended to supplement and not replace the qualitative estimates by a qualified and experienced professional. The quantitative input includes the RQD and the joint condition ($JCond_{89}$). Similar to the GSI, the $JCond_{89}$ value is based on a qualitative evaluation of the discontinuity surface and other features, including persistence, aperture, roughness, infilling, and weathering (Hoek et al., 2013, Reference 8.14). Alternatively, the $JCond_{89}$ may be estimated from a reduced set of estimates known as the joint roughness number (Jr) and joint alteration number (Ja) as part of the Site Investigation Program following Hoek et al. (2013, Reference 8.14). The semi-quantitative relationships for GSI and $JCond_{89}$ from Hoek et al. (2013) are provided below:

$$\mathrm{GSI} = 1.5\mathrm{JCond}_{89} + \frac{\mathrm{RQD}}{2} \qquad (3\text{-}1)$$

where: $$\mathrm{JCond}_{89} = 35\frac{\left(\frac{\mathrm{Jr}}{\mathrm{Ja}}\right)}{\left(1+\frac{\mathrm{Jr}}{\mathrm{Ja}}\right)}$$

As described in RG 1.132 (Reference 8.64), characterization of the shear strength for planar discontinuities, such as bedding planes, faults, fracture zones, joints, and shear zones typically include laboratory testing of subsurface discontinuities recovered from samples (e.g., direct shear and triaxial compressive strength tests), tests on saw cut portions of recovered rock cores to determine base friction angles, or, less commonly, in-situ tests of the discontinuities under specific loading conditions (e.g., in-situ direct shear tests). In-situ direct shear tests at nearby outcrops or in shallow rock excavations can be considered. Because the most common method is testing

recovered subsurface samples, empirical corrections are required for surface roughness, intact surface strength, and the scale of the tested sample (e.g., Barton-Bandis criterion). When empirical corrections are used, the Site Investigation Program will need to determine estimates of appropriate parameters (e.g., the joint roughness coefficient [JRC] and joint wall compressive strength [JCS] for the Barton-Bandis criterion [ISRM, 1978]).

When the rock discontinuities are filled with another material, the shear strength may decrease or increase depending on the type of infill material. Testing of the infill material is required when there is a significant thickness of weaker material that may control the strength of the discontinuity. When a nonlinear relationship between shear strength and normal stress (e.g., Barton-Bandis criterion) is not desired, the equivalent friction angle and cohesion may be determined from the tangent to the nonlinear relationship for the shear strength of planar discontinuities.

Cavities in the rock mass from karst or dissolution may decrease the effective rock mass modulus and create a highly variable interface between the rock and overburden. The presence of cavities should be identified during the subsurface investigation. Consistent with RG 1.132 (Reference 8.64), the spacing and depth of investigation locations should be reduced to detect the anticipated features.

A grouting program may be required to fill cavities and control seepage. The grouting program should include the potential to remove infilling from cavities using a water wash and fill the cavities as much as possible with grout. Replacing infill or open cavities with grout should increase and control variations in the rock mass modulus around and beneath the structures. Contact grouting is also required after construction of the shaft to avoid irregular external loading from voids – natural or due to overbreak during construction – on the exterior of the shaft. The rock surface may require modification through excavation or ground improvement to avoid significantly different stiffness along the shaft. Epikarst may form pinnacles or similar features that may result in variable stiffness along the shaft near the bedrock and overburden interface. The effect of potential cavities in the rock mass and variations at the bedrock and overburden interface on shaft deformation are evaluated on a site-by-site basis.

3.2 Construction Inspection and Testing Program

3.2.1 Excavation and Foundation Inspections and Testing

Excavation and foundation inspections and testing programs are implemented for the BWRX-300 that meet the geotechnical and foundation requirements of the NRC Inspection Manual 88131 (Reference 8.15), including:

R. Key Site Parameters are verified by checking if the required values for average allowable static bearing capacity and maximum allowable dynamic bearing capacity for normal plus SSE loading have been met at the excavation depth.

S. Soundness of the exposed rock is checked by qualified personnel to confirm the results of rock mass characterization described in Section 3.1.3. This includes visual inspection and testing of:

- Rock material properties, such as rock type, color, particle size, hardness, and strength.

- Rock mass properties, such as rock structure, shear strength, deformation modulus, hydraulic conductivity, and attitude.

The additional geotechnical borings and borehole televiewer locations at the perimeter of the BWRX-300 RB shaft are intended to provide compensatory data if there is limited access for the excavation and foundation inspections and testing programs due to excavation support, rock reinforcement, surface treatments, and/or waterproofing. This additional data is intended to provide sufficient information to characterize inaccessible portions of the excavation.

When the rock excavation is accessible, additional in-situ testing can also be completed for rock mass properties such as shear strength and deformation modulus. These tests may include larger-scale plate loading of the in-situ rock mass in the excavation or in-situ direct shear tests on discontinuities exposed by the excavation using appropriate methods from RG 1.132 (Reference 8.64). Overcoring with a borehole deformation gauge in the rock excavation can also be implemented to confirm the in-situ stress field at some sites.

3.2.2 Building Structure Construction Inspections and Testing

The BWRX-300 RB construction inspection and testing program satisfy the structural concrete activities requirements of the NRC Inspection Manual 88132 (Reference 8.16) and structural welding inspection requirements of NRC Inspection Manual 55100 (Reference 8.17). The program includes:

- The visual surface inspection acceptance criteria that include quantitative limits for the appearance of leaching or chemical attack, pop outs or surface voids, scaling, spalling, corrosion staining, settlements, and cracks.
- ACI 349.3R guidance (Reference 8.18), which is recommended by ASME XI Rules for Inservice Inspection of NPP Components, Subsection IWL for visual inspections of exposed surfaces. ACI 349.3R requires that accessible concrete surfaces do not have voids greater than 2 inches; scaling is limited to 8 inches in diameter and 0.75 inches in depth; and cracks are limited to widths of 0.04 inches or smaller.
- ASME XI, Subsection IWL 1220 (b) and (d) exempts concrete surfaces that are covered by a liner or adjacent to a foundation or backfill from detailed visual inspections.
- Concrete surfaces exposed to soil, backfill, or groundwater are evaluated to determine susceptibility of the concrete to deterioration and the ability to perform the intended design function under conditions anticipated until the structure no longer is required to fulfil its intended design function. The evaluation includes the following:
  a) Existing subgrade conditions, including groundwater presence, chemistry, and dynamics; aggressive below-grade environment, or other plant-specific conditions that could cause accelerated aging and degradation.
  b) Existing or potential concrete degradation mechanisms, including, but not limited to, aggressive chemical attack, erosion and cavitation, corrosion of embedded steel, freeze-thaw, settlement, leaching of calcium hydroxide, reaction with aggregates, increase in permeability or porosity, and combined effects.

Table 3-4: Degradation Conditions and Criteria for Accessible Steel Structures

Degradation Condition	First-Tier Criteria	Second-Tier Criteria
Corrosion and/or corrosion stains	Absence of condition(1) (2)	Condition present, but determined acceptable after further review(3) (4) (5)
Bulges or depressions in liner plate	Absence of condition(1)	Condition present, but determined acceptable after further review(3)
Cracking/degradation of base or weld metal	Absence of condition(1)	Condition present, but determined acceptable after further review(3)
Leakage/Seepage (presence of water)	Absence of condition(1)	Condition present, but within original design limits of active leak-detection system and the leaking material and source do not present any adverse consequences(3)
Detached embedments or loose bolts	Absence of condition(2)	Condition present, but determined acceptable after further review (4)

(1) Section 5.1.2 of ACI 349.3R (Reference 8.18)

(2) Section 5.1.3 of ACI 349.3R (Reference 8.18)

(3) Section 5.2.2 of ACI 349.3R (Reference 8.18)

(4) Section 5.2.3 of ACI 349.3R (Reference 8.18)

(5) Section IWE-3500 of ASME XI (Reference 8.20) provides a threshold of 10% loss of nominal wall thickness.

3.4 Field Instrumentation Plan

Field instrumentation that beyond the current regulatory guidelines, is deployed to monitor the magnitude and distribution of pore pressure and amount of deformation during excavation, construction, loading and continuing through the BWRX-300 plant operation. The instrumentation provides recordings that can frequently be benchmarked against design estimates. Short-term and long-term settlement monitoring plans are developed that can detect both vertical and horizontal movements in and around the structures, as well as differential distortion across the foundation footprint and differential settlements between the CB, TB, RwB and RB foundations.

The specific locations of the sensors are dictated by the subsurface conditions and areas identified in the design where maximum stress, strain, and pore pressures are anticipated along the perimeter of the shaft. The definitive number of instruments is established during design stages of the monitoring system considering that the field instrumentation system shall be capable of:

T. Measuring the rate of heave during excavation, especially at the end of excavation and at the bottom center and edges of the shaft.

U. Measuring the rate of lateral displacement of excavation walls, throughout its depth, during and at end of excavation.

V. Measuring the distribution of pore pressures around and below the RB shaft.

W. Measuring the total settlement and tilt of the RB shaft, during construction, loading, and operation; this will require deploying a system of sensors and survey monuments throughout the perimeter of the shaft at bottom, medium depth, and plant grade.

X. Measuring settlement of the auxiliary and surrounding structures of the BWRX-300.

Figure 3-3 indicates the required implementation period that the field instrumentation has to accommodate. Some instruments will be temporary while others are permanent. Some instruments, such as piezometers, are installed prior to excavation. Installation for extensometers or other survey monuments are to be taken at the appropriate stage of the BWRX-300 life.

To achieve the required monitoring capabilities, the field instrumentation consists of four primary elements:

1. Piezometers to measure pore pressure distribution. Vibrating wire piezometers are preferred for this purpose as they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis. Piezometers should be screened at elevations that are representative of the site-specific hydrogeologic conditions.
2. Settlement monuments placed directly on concrete, preferably on the corners of the structures at grade that are accessible with conventional surveying equipment.
3. Settlement sensors and extensometers used for settlement prone soils or deformation prone rock masses.
4. Earth pressure sensors to monitor vertical and lateral pressure along the walls of the shaft.

For deployment in soft soil conditions, settlement sensors are installed within a borehole attached by a Borros anchor as described in Reference 8.21. For hard soil and rock conditions, sensors may consist of rod type extensometers anchored below loading points. The borehole extensometer includes anchors, extension rods and a reference head. The anchor is connected to the head of the instrument by extension rods typically placed within a protective sleeve. This sleeve ensures that the rods can move freely and translate all movement of the anchor to the tip of the rod. The movement of the rock or soil mass relative to the head can then be calculated by measuring the displacement of the tip of the extension rod to a reference plate located in the head of the extensometer as the one described in Reference 8.22. The instrument can be used to measure deformation of laterally loaded retention walls and to monitor settlement in foundations.

The need for direct monitoring of interfaces will be a site-specific determination depending on their importance and the specific subgrade conditions.

The groundwater levels at the site are monitored using pressure transducers installed in multiple screened wells installed across the site. This data provides groundwater elevations, groundwater flow direction(s) and groundwater gradients. This information is used during excavation and construction for estimating seepage rate, short-term dewatering rates, and effective stresses under static and dynamic conditions.

When practical and applicable, sensors are connected to a datalogger(s) programmed to read the sensors periodically. Some of these sensors are installed in cased boreholes and the sensors can be removed, maintained, or replaced during the needed phases of the project. Other sensors, such as the earth pressure sensor, need to be buried in the subsurface and cannot be removed or replaced once backfilled. Such sensors are installed with redundancy to monitor the necessary data for the specific duration of the project phase when such data is used.

HH. Structural modeling of the main civil/structural components of the BWRX-300 and auxiliary facilities, described in Section 4.3.2, along with varying live and dead loads throughout the construction process.

II. Fluid Soil Interaction, described in Section 4.3.3, to capture an adequate distribution of the space and time variation of pore pressures.

JJ. BWRX-300 life stages: siting, excavation, construction, loading, and operation described in Section 4.3.4.

4.3.1 Interface Models

4.3.1.1 Interfaces Between the Structures and the Subgrade Media

The behavior of the contact at the base might not be critical for the RB because sliding and overturning are likely controlled by the deep embedment. However, the behavior of contact between the walls and soil, influences the soil pressures exerted on the structure along its embedded depth. The contact behavior depends on the selected construction methodology and changes through construction. For example, the contact condition of the BWRX-300 RB outer wall, when poured using a slurry wall or rock face as formwork, is different than the contact gained from a typical construction and backfill/grouting process. Figure 4-1 provides a schematic showing interfaces between structure and the surrounding media.

The interface is modeled, as is the case for the soil, with the use of an elastoplastic relationship based on an elastic deformation modulus and shear resistance. Figure 4-2 shows an example of interface rheologic modeling typically used for BWRX-300 FIA. A series of spring couplers are simulated at the connecting grid points at the interface. Each spring is represented by an elastoplastic model with Mohr-Coulomb criterion for shear failure.

When interface elements are used to represent the structure and soil/rock interaction, node pairs are created at the interface. From a node pair, one node belongs to the structure and the other node belongs to the soil/rock. The relative displacements (i.e. slipping/gap opening) can be simulated through elastic-perfectly plastic springs between these two nodes. Typically, two sets of springs are used for interface elements. One elastic-perfectly plastic spring to model the gap displacement and one elastic-perfectly plastic spring to model slip displacement. The simulation of gaps opening between the structure and soil/rock can be achieved through activating a tension cut-off for the spring that does not allow any tension at the interface.

The parameters of the slipping spring can be taken from the material set of the adjacent soil/rock elements based on the results from the Site Investigation and Laboratory Testing programs as described in Section 3.1. A strength reduction factor can be used to adjust the spring stiffness based on the roughness of interaction and soil/rock residual strength when the sliding occurs. It is also possible to assign strength properties to interface elements based on direct measurements. If planar geosynthetic products are used during construction of the wall, shear properties are assigned to the interface elements representative of shear properties at geosynthetic/soil interfaces.

As is the case for soil and rock material constitutive models, the use of complex modeling capabilities for modeling interfaces introduces the challenge of identifying adequate input physical parameters. To address the uncertainties in these input parameters in a conservative manner, the analysis may be conducted using bounding limits for the rheologic elastoplastic models assigned

operation. The model can simulate short-term as well as long-term dewatering or pumping as dictated by field conditions. The model simulates the changes in pore water pressures of the soil in response to unloading during the excavation stage and loading during construction and loading stages.

4.3.4 Analysis Staging Approach

Section 3.2 provides a description of the life stages of the BWRX-300, starting from the site investigation and ending with the plant operation. The BWRX-300 FIA are performed on numerical models that have the features to perform an integrated analysis of the stress, and deformation fields for each of the identified life stages:

4.3.4.1 Site Characterization

The FIA begins with the site itself, in its native condition, prior to any excavation or construction activities. During this stage, the initial stress conditions are aligned with the initial baseline displacement field. Initial stress conditions include, if applicable, the influence of groundwater aquifers and residual horizontal stresses.

4.3.4.2 Excavation

During the BWRX-300 RB shaft excavation, shown on Figure 4-4, soils and rock around and below the shaft may experience tensile stresses. The selected constitutive models allow for expansion response of soils resulting in heave or added pressures on excavation support structures. The changes in site conditions made prior or during the excavation are introduced in the FIA model following the sequence of the excavation plan. Non-linear interfaces are modeled between stabilization walls and soil.

As shown on Figure 4-5, the excavation simulation resembles the scheme planned for the specific site, by staging the removal of soil layers as excavation progresses and excavation support and site improvements are made. The stability of the excavation is verified in analytical space and later compared against field observations. The process allows for the design and monitoring of a safe excavation.

At the end excavation, the stress and displacement fields of the surrounding media, as well as the distribution of pore pressure, will have evolved. The "after excavation" condition is used as the initial condition for the analysis of the construction stage.

below grade RB shaft exterior wall. The results obtained from the contact spring elements serve to:

- validate the earth pressure loads considered by the design as described in Section 5.1.3, and
- determine whether separation between RB shaft wall and soils occurs in the static and dynamic loadings as discussed in Section 5.3.9.

The mesh of the FE models is sufficiently refined to produce stress demand calculations that are not significantly affected by a further refinement of the FE size or the shape. Finer meshes are used around penetrations and openings that are larger than half of the wall or slab thickness. Meshes of major walls and slabs consists of at least four shell elements along the short direction and at least six shell elements along the long direction.

The FE models used for seismic SSI analyses have a sufficiently refined mesh to be capable of transmitting the entire frequency range of interest for the seismic design of the RB SSCs. In accordance with the requirements of ASCE\SEI 4-16 (Reference 8.7), Section 5.3.4, the FE mesh shall be smaller than or equal to one-fifth of the smallest wavelength transmitted through the soil model, i.e. the maximum mesh size:

$$d_{max} \leq \frac{V_S}{5\, f_{cutoff}} \quad (5\text{-}1)$$

where: V_S is the shear wave velocity of the transmitting soil material; and

f_{cutoff} is the cutoff frequency of analysis determined as described in Section 5.3.2

Larger element sizes may be used when justified as described in Section 5.3.4 of ASCE\SEI 4-16. Stiffness properties are assigned to structural members in the RB FE model in terms of Young's modulus and Poisson ratio that are determined in accordance with the governing design codes:

- American Concrete Institute ACI-349-13 (Reference 8.24) for the reinforced concrete members; and
- AISC N690-18 (Reference 8.25) for the steel and steel-plate composite (SC) members.

5.1.2 Soil-Structure Interaction Modeling Assumptions

Several simplified assumptions are introduced in the SSI design analyses of RB FE model to enable an efficient calculation of stress demands on the RB structure due to pressure loads from soil and rock surrounding and supporting the RB shaft. The following are the main SSI modeling assumptions:

1) The properties of the subgrade materials are assumed isotropic and linear elastic;
2) The non-linearities at soil-structure interfaces are neglected;
3) The rock mass is assumed continuous and the presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones is neglected;

4) The static lateral pressures on the RB shaft due to the weight of ~~rock is assumed~~ self-supporting rock, i.e., excavated rock that does not require ~~no~~ lateral support ~~is required of the excavated rock~~ can be neglected.

As described in Section 5.2.1, an approach is used for the development of linearized properties of soil and rock materials for the 1-g static SSI analysis to provide upper bound estimates of the demands on the RB structural members. Upper bound structural deformations and stress demands and lateral soil pressures on the RB below-grade exterior walls are estimated by using upper bound values for the soil unit weight and soil and rock Poisson's ratio paired with lower bound values of soil and rock elastic moduli.

The following stiffness properties are assigned to the contact springs at the SSI interfaces in the RB FE model for 1-g design analysis to provide upper bound lateral soil pressures on the RB below-grade exterior walls:

- The contact springs in the direction normal to the RB exterior walls are assigned properties representing upper bound stiffness conditions at the SSI interfaces; and
- The friction at the RB exterior walls is neglected by assigning very low stiffness properties to the contact springs in vertical and tangential direction.

The soil and rock strata in the SSI models used for calculating demands for design of RB structure are modeled based on the principles of continuum mechanics using isotropic linear elastic properties. Possible fracture zones, joints, bedding planes, discontinuities and cavities in the rock are not explicitly included in the design SSI analyses models. The stiffness properties assigned to the rock materials are developed, as described in Section 5.2.1.2, using empirical engineering and geomechanical rock mass classifications that quantitatively characterize the geologic and engineering parameters of rock masses.

The approaches described in Section 5.2.1.2 to calculate the equivalent linear properties of rock are applicable to structures that are relatively large compared to the block size of the rock mass and assumes the closely spaced discontinuities have similar characteristics where isotropic behavior of the rock mass is valid. For site-specific conditions where rock mass behavior may be highly anisotropic, adjusted rock Young's elastic modulus and Poisson's ratio are used as input for the design analysis to obtain conservative design demands that include adequate margins to envelope the effects of possible anisotropic behavior of rock masses. The equivalent linear properties of the rock mass may be adjusted based on the results of the FIA performed using constitutive models that can capture the important aspects of the anisotropic and non-linear behavior of the rock mass as described in Section 4.2.2. The adequacy of the design margins is evaluated based on the levels of uncertainty present in the determination and modeling of the site-specific subgrade conditions.

When the discontinuity spacing is large compared to the dimensions of the excavation, the potential for unstable blocks or wedges and swelling or squeezing rock units need to be evaluated. The evaluation of the potential loads on the RB from rock blocks and wedges may be completed using results obtained from:

- FIA models that include the rock discontinuities, as the ones described in Section 4.3.1.2, and/or

- simple static or pseudostatic force limit equilibrium analysis models as the one shown in Figure 5-1. ~~A simple example of a model for force equilibrium analysis of rock stability is provided in Section 5.1.4.3.~~

Equivalent lateral coefficients may be calculated using upper bound lateral pressure estimates obtained from the FIA or limit pseudostatic force equilibrium analyses. Equation (5-14) is then used to calculate Poisson's ratio values for the rock mass having the potential for unstable blocks or wedges. These Poisson's ratios and upper bound unit weights are assigned to the rock mass to ensure the design adequately addresses the potential for unstable rock blocks or wedges.

Strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness will frequently be self-supporting even if some reinforcement is required to ensure a safe excavation. Typically, rock masses will yield slightly during construction – even with well-placed reinforcement – and arching will reduce the lateral loads except in highly fractured, weak, swelling, or squeezing rocks. ~~Joints and other weak planes may create isolated blocks that are unstable; however, these blocks are not typically large relative to the area of the structure and would be unlikely to produce significant loads on the exterior of the structure compared to other loads (e.g., hydrostatic). These blocks would also not be able to create a cascading failure once the structure is in place.~~

Because it is much more economical to reinforce the rock mass than to support it, rock reinforcement is used to create a self-supporting rock mass when the natural rock mass is not self-supporting. Reinforcement like tensioned and untensioned anchors may be installed inside the rock mass to help the rock mass support itself by eliminating progressive failure along planes of low strength as described in USACE 1110-1-2907 (Reference 8.26). Frequently, the reinforcement addresses specific rock wedges (keying) or is designed to form a beam or arch within the rock to create a stable, self-supporting excavation. Surface treatments such as shotcrete, strapping, and mesh may also be used for stabilization, protection of exposed rock, and control of loosened rock.

The design of the BWRX-300 typically considers this rock reinforcement as initial ground support that is separate from the permanent ground support system because the rock reinforcements and any surface protection may be inaccessible after construction. Therefore, the design addresses the rock loads remaining after the initial ground support degrades by including the potential weight of the solid rock in the design 1-g SSI analysis based on the results of non-linear FEA and/or limit equilibrium analyses as described in Section 5.1.3.

The SSI analysis of RB FE model are performed for a set of subgrade profiles to account for the variability and uncertainties in the subgrade material properties in accordance with the regulatory guidance of SRP 3.7.2 Subsection II.4 and ASCE/SEI 4-16 (Reference 8.7), Section 5.1.4. To address the effects of primary non-linearity, soil dynamic properties are used that are compatible to the free-field strains generated by a typical design level earthquake. These strain-compatible properties are developed as described in Section 5.2.4.

The effects of secondary non-linearity induced in the soil and rock by the structural vibration are neglected because in general, the structural vibration induces plastic deformations of the soil and dissipation of energy in the SSI system that reduces the structural response as shown in Reference 8.27 and Reference 8.28. On the other hand, the secondary non-linearity of subgrade

materials may amplify the magnitude of the dynamic lateral pressures. The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones within the rock mass may also affect the stability of individual blocks or the rock mass during an earthquake that can potentially amplify the seismic rock pressure loads. Section 5.3.11 describes the approach used to evaluate the effects of subgrade materials non-linearity on the seismic response and design BWRX-300 RB when it is constructed at sites characterized with a high non-linear behavior of the subgrade materials and high seismicity. To account for possible amplifications of dynamic earth pressures on the RB shaft due to rock discontinuities, fracture and weak zones, the dynamic stiffness and unit weight properties of the rock masses may be adjusted using the same approaches as the ones used for adjusting the static rock properties.

The design basis seismic analyses of BWRX-300 RB are performed on models that assume fully bounded conditions at the interfaces between the RB structure and the subgrade. Depending on the subgrade conditions and the intensity of the design ground motion, separations may occur at the SSI interfaces during an earthquake event. Section 5.3.9 describes a conservative approach for addressing these effects of soil separation on the RB seismic response and design.

5.1.3 Design Earth Pressure Load Validation

Section 4.0 describes the FIA performed on numerical models representative of the non-linear constitutive behavior of soil and rock materials surrounding the RB shaft and employ non-linear interface modeling features capable of capturing the effects of non-linearities at the soil-structure contact surfaces. The model also includes the main structural elements of the RB that adequately represents the stiffness properties of the structure interacting with soil and accurate calculations of the contact pressures at soil-structure interfaces.

Results for maximum soil and rock pressure loads on the RB exterior walls obtained from the FIA and the linear elastic 1-g design analysis are compared to:

- assess the effect of non-linear and anisotropic behavior of subgrade materials on the soil and rock pressure demands;
- demonstrate that the SSI modeling assumptions listed in Section 5.1.2 yield conservative design demands; and
- assess the conservativism of the soil and rock pressure demands obtained from the 1-g design analysis for the design of RB structure.

As described in Section 4.3.4, the FIA considers staged excavation, construction and loading sequences to adequately model the change in in-situ stress due to construction activities and establish the initial conditions for calculation of soil pressures at the stage when the plant is in operation. However, detailed stages of excavation and construction as presented in Section 4.4 are not required for the soil and rock pressure loads validation. Stages like excavation and construction may be completed in a single step instead of multiple steps because the monitoring details are not required.

The validation of soil and rock pressure loads may consider the subgrade improvements like consolidation grouting, rock reinforcement, and soil support made during the construction. However, these improvements are typically considered only as initial ground support that is

separate from the permanent ground support system because these types of reinforcements and any surface protection will be inaccessible for monitoring and repair after the construction. Therefore, unimproved soil and rock conditions are considered due to the uncertainty in:

- the long-term durability of grout, as noted in Paragraph 2-5 of USACE EM 1110-2-3506 (Reference 8.29);
- potential degradation of rock reinforcement, as noted in USACE EM 1110-1-2907 (Reference 8.30); and
- degradation of other soil support system.

This additional rock load on the RB shaft wall may be considered uniform with contact grouting to avoid stress concentration or point load associated with the block or wedge that is reinforced to stabilize the rock excavation. The evaluation of these rock pressure loads assumes that the excavation has reached stability with initial rock support and that the liner will accept 100 percent of the initial rock support as it relaxes over the lifetime of the structure. The~~se~~ initial rock support loads should be conservative because rock loads in stressed rock masses are typically not following (e.g., they ~~are not independent of displacement and~~ typically reduce with displacement due to arching). The notable exception would be due to the presence of hydrostatic loads and swelling or squeezing rock displacements that are time dependent and can ~~may~~ continue to apply a large load with continued displacement. When pressures from swelling or squeezing rock displacements create the potential for loads on the RB shaft, the RB shaft can be designed to take the full pressure, a compressible material can be placed between the RB shaft and rock to reduce the pressures, or a scheduled construction delay can be added to allow the deformation to occur and reduce the pressures. If the degradation of initial support for large rock blocks potentially creates unacceptable high pressures, other options can include overexcavating and backfilling the rock block to reduce the potential pressures, the use of degradation resistance rock reinforcement to permanently support the rock block, or changes in the BWRX-300 location to improve the relative position of the rock block and reduce the potential pressures on the RB shaft.

The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. This potential load transfer is dependent on the geometry of the discontinuities, surface structure and embedded structure. When the additional load from the surface structure may be transferred to a potentially unstable rock block or wedge, this additional load should be included in the determination of reinforcement and the potential rock load on the exterior of the shaft or the rock block or wedge may be over-excavated and backfilled to reduce the load. Consideration of the geometry of the load transfer may allow the surface structures to be re-arranged to reduce or eliminate this load transfer to a potentially unstable rock block or wedge.

If cavities are present at the deployment site, sensitivity analysis are also performed by varying locations and sizes of cavities to address the effects of potential cavities on the rock pressure demands on the RB structure during operation.

The pressure load validation FIA uses the constitutive models described in Section 4.2 to represent the non-linear response of soil and rock subgrade materials, and the models described in Section 4.3.1 to represent the response at interfaces including the interfaces of RB structure with the surrounding subgrade. Because the intent of the FIA is to calculate best estimates of the soil

and rock pressure loads, constitutive and interface models are developed using best estimate soil and rock properties obtained from the results of site investigation and laboratory testing programs described in Section 3.1. The stiffness of the RB structure in the FIA models is calculated per the governing design codes. Conservative design values obtained from the literature can also be used for certain input parameters.

A best estimate soil and rock pressure profile on the RB shaft is developed as an envelope of all maximum lateral pressure values calculated by the non-linear FIA of all analyzed post-construction stages and scenarios. This lateral pressure profile is compared to the lateral pressure profile developed from the results of the linear elastic 1-g design analysis to confirm the equivalent linear elastic model provides adequately conservative loads for the structural design. Soil and rock design pressure margins are calculated based upon the minimum values and the distribution of the ratio between the design soil and rock pressures obtained from the 1-g linear elastic analysis and the best estimate pressures obtained from the non-linear FIA. If the values of the calculated soil and rock design load margins are below the values deemed adequate to address the uncertainties and variations of subgrade properties, the rock mass weight or the equivalent linear soil and rock stiffness properties used for the 1-g design analysis are adjusted following the recommendations in Section 5.1.2. Adequate values of the soil and rock design load margins are established based on the uncertainties and variability of soil and rock properties used as input for the non-linear FI~~E~~A and the significance of the non-linear and anisotropic response of subgrade materials on the soil and rock pressure demands. Since the earth pressure load is the most important site-related load affecting the structural integrity of the deeply embedded RB structure, the validation of the earth pressures often is sufficient to address the uncertainties in the important site subgrade parameters that can affect the design of the RB structure. For sites where the consideration of earth pressures alone cannot completely address all uncertainties related to the subgrade stiffness parameters, the adequacy of the design may be demonstrated by comparing the non-linear FIA and the design 1-g SSI analysis displacement results at the interfaces of the RB structure with the surrounding soil and rock. Alternatively, the FIA calculated displacements may be applied to the RB structural design analysis model as boundary conditions at the soil-structure interfaces to calculate stress responses of selected major RB structural members that are in line with the FIA calculated subgrade response. These FIA compatible structural stress results are then compared to the corresponding results of the design 1-g SSI analysis to demonstrate that the design adequately address the uncertainties related to the determination of subgrade stiffness parameters.

If the results of non-linear static FIA indicate that the non-linear and anisotropic effects have a significant effect on the rock soil pressures and the site is characterized by a high seismicity, sensitivity SSI analyses are performed on non-linear models, as described in Section 5.3.11, to assess the effects of non-linear soil and rock response on the dynamic lateral pressure demands.

5.1.4 Probabilistic Earth Pressure Analyses

Probabilistic analyses may be performed to demonstrate that the magnitude of earth pressures used for the design are adequate to address uncertainties in the pressure load calculations. The external wall of the RB that is contact with soil is subdivided into discrete regions. The general approach consists ~~on~~ of computing the probability density function of the subgrade pressure at each discrete region to calculate the probability distributions of soil and rock pressure loads on the RB

below-grade exterior walls and using them to determine the probability for the earth pressures on the RB shaft to exceed the design pressure loads calculated by the 1-g SSI analysis.

The probabilistic earth pressure load analysis addresses two types of uncertainties in the calculations of earth pressure loads:

- Parameter uncertainties related to natural randomness ~~and uncertainties in measurements of mechanical properties~~ of in-situ subgrade materials, errors in their measurements and uncertainties related to development of site subgrade parameters; and
- Model uncertainties related to the simplifying modeling assumptions discussed in Section 5.1.2~~models used for earth pressure calculations~~.

Parameter uncertainty includes random variability of measured parameters including spatial variability and systematic measurement errors as well as uncertainties related to the methods used for the development of site subgrade parameters from empirical relationships using directly measured or indirect estimated site subgrade parameters. The random variability is manifested as the scatter of the data around a mean trend and is composed of the spatial variation of the subgrade properties and random measurement errors. Because the random measurement errors are often not distinguishable from spatial variation of the subgrade properties, they are usually considered jointly. Systematic error is divided into:

- Statistical error in the mean that can be reduced with increasing the sample size and number of measurements and tests being performed
- Bias in sampling and measurement procedures that is corrected by means of correction techniques/algorithms
- Bias introduced by the methods used for development of site subgrade parameters that is addressed by considering different approaches and empirical equations to calculate discrete probability distributions that are then combined as described in Section 5.1.4.4.

The model uncertainty that represents the uncertainty related to the model's ability to accurately predict the soil and rock pressures is manifested as a bias error in the earth pressure calculations. In general, the model uncreate is reduced by using more sophisticated models and an increasing number of model parameters. On the other hand, the increasing number of parameters used in the sophisticated models increases the parameter uncertainty and may reduce the overall confidence in the calculated soil pressure results. The model uncertainty is approached by means of considering different models that utilize fewer input parameters resulting in discrete probability distributions that are combined as described in Subsection 5.1.4.4.

As noted in Section 5.1.3, the structural stress results compatible to the deformations at the RB interfaces with the surrounding soil and rock may be used to address the uncertainties related to the modeling of subgrade stiffness parameters. For sites where probability calculations may be required to demonstrate that the available design stress margins are adequate, the probability of the structural stresses responses to exceed the stress demands used for the design of the RB structure may be obtained from probabilistic analyses performed using simplified FIA models and following the approaches described in this section.

Table 5-1: Models for Probabilistic Earth Pressure Analyses

Subgrade Type	Site Parameter (x_i)	Model
soil	unit weight	Analytical equations
	cohesion	
	friction angle	
rock	rock mass properties	Force limit equilibrium, FE or a finite difference model
	unit weight	
	cohesion	
	friction angle	
	weak zone orientation	
	weak zone area	

Simple models that do not require explicit calculations of the state of strain and stress in the ground materials, are used for the probabilistic analyses of earth pressures on the RB shaft in contact with subgrade materials which mechanical properties are assumed to be continuous. For example, the following three models can be used to calculate lateral earth pressure coefficients representing three possible states:

a. at-rest condition representing essentially no movement of the structure relative to the surrounding subgrade;

b. active condition when the structure moves away from the surrounding subgrade; and

c. passive condition when the structure moves towards the surrounding subgrade.

These simple models provide probabilistic earth pressure distributions from the probabilistic distributions of the basic subgrade material strength parameters, the internal friction angle (φ), the cohesion (c) and the friction angle (φ_w) between the subgrade and RB cavity wall.

Force equilibrium models are used for probabilistic analysis of rock masses with discontinuities that may control the stability of individual blocks or the rock mass when the orientation is disadvantageous. Depending on the geometry of the discontinuities relative to the free face of the excavation, one or more blocks may slide along the discontinuities.

As shown on Figure 5-1, the sliding of the rock block driven by the surcharge load and its own weight is resisted by:

- the resistance force along the rock discontinuity due to cohesion (c_d) and the friction represented by the friction angle (φ_d); and
- the resultant of pressure loads at the rock-structure interface.

where: E_M is the Menard's modulus calculated directly from the pressuremeter field measurements of soils under drained conditions; and

α Menard's correction factor.

Menard's α factor is applied to correct the E_M that usually underestimate the stiffness of the soil because it is developed from stress-strain measurements over a large range of strains assuming infinite borehole and uniform soil properties that remain undisturbed by the testing probe. Menard's α factor are determined empirically for different soil types and range from 0.25 to 1 according to Reference 8.37.

Table 5-2 provides examples of empirical correlations published in the literature for calculations of E_{st} of different types of soil materials from SPT and CPT results.

The following theory of elasticity equation is used to calculate ν_{st} values representative of soil at-rest (K_0) lateral pressure conditions:

$$\upsilon_{st} = \frac{K_0}{1 + K_0} \tag{5-14}$$

The BWRX-300 design considers upper bound values for at-rest coefficient K_0 to address uncertainties and variations of subgrade properties. The K_0 values are determined based on the results of site investigations and laboratory testing programs described in Section 5.2.4. Using measurements of effective angle of friction (ϕ_s), K_0 values for normally consolidated soils may be determined from the following simplified Jacky's equation:

$$K_0 = 1 - sin\ (\ \phi_s) \tag{5-15}$$

K_0 values for over-consolidated materials (e.g. stiff to hard clays) may be determined from the following modified Jacky's equation:

$$K_0 = [1 - sin\ (\ \phi_s)]\ OCR^{sin\ (\ \phi_s)} \tag{5-16}$$

where *OCR* is the over-consolidation ratio.

5.2.1.2 Rock Mass Equivalent Linear Properties

Equivalent linear E_{st} of rock masses can be estimated based on the intact rock Young's Modulus (E_{ri}) and the rock mass classification determined from results of the site investigation program. The following Hoek and Diederichs (Reference 8.23) equation may be used to adjust the intact rock E_{ri} and calculate rock mass E_{st} based on the rock mass Geotechnical Strength Index (GSI):

$$E_{st} = E_{ri}\left[0.02 + \frac{1-0.5D}{1+e^{\left(\frac{60+15D-GSI}{11}\right)}}\right](\text{GPa}) \tag{5-17}$$

where: E_{ri} and E_{st} are in units of giga Pascals (GPa); and

D is the degree of rock disturbance which values range from 0 for undisturbed confined rock to 1 for blast damaged rock in a typical open pit mine slope.

The following equation from Reference 8.21, which was developed by Galera, Alvarez, and Bieniawski, may be used to estimate rock mass E_{st} by adjusting the measured intact rock E_{ri} using its Rock Mass Rating (RMR) qualification:

$$E_{st} = E_{ri}\, e^{\left(\frac{RMR-100}{36}\right)} \text{ (GPa)} \tag{5-18}$$

where: E_{ri} and E_{st} are in units of GPa.

Results of UC strength laboratory tests performed on intact rock specimens can serve as the basis for development of E_{ri} values. Reliable, measured values of E_{ri} are often difficult to obtain due to sample damage from micro-cracking in recovered rock samples. The strength measurements obtained from UC strength tests are often considered more reliable because the sample damage has a greater effect on E_{ri} than on the UC strength. More reliable values of E_{ri} for use in Equations (5-17) and (5-18) can be obtained from the UC strength measurements as follows:

$$E_{ri} = MR \text{ (UC strength)} \tag{5-19}$$

where: MR are modulus ratio values like those provided in Table 3 of Reference 8.23 for various rock types and textures.

If UC strength measurements of intact rock E_{ri} are not available, the following equation proposed by Hoek and Diederichs in Reference 8.23 may be used to estimate E_{st} of the rock mass in GPa based solely on its GSI:

$$E_{st} = 100\left[\frac{1-0.5D}{1+e^{\left(\frac{75+25D-GSI}{11}\right)}}\right] \text{(GPa)} \tag{5-20}$$

where: D is the same rock disturbance parameter as the one used in Equation (5-17).

Empirical equations may be used to estimate E_{st} of the rock mass in GPa based on its RMR qualification. The following equation proposed by Serafim and Pereira in Reference 8.38 may be used to calculate rock mass E_{st} for values of RMR < 50:

$$E_{st} = \left[10^{\left(\frac{RMR-10}{40}\right)}\right] \text{ (GPa)} \tag{5-21}$$

The following equation proposed by Bieniawski in Reference 8.11 may be used to calculate rock mass E_{st} for values of RMR < 50:

$$E_{st} = [2(RMR) - 100] \text{ (GPa)} \tag{5-22}$$

Upper bound ν_{st} values for rock masses may be developed based on V_P and V_P measurements and the level of rock fracturing. It is anticipated the ν_{st} values developed based on V_S and V_P measurements will typically be higher than or similar to measurements on recovered rock samples due to the rock sample damage. For most rock masses, ν_{st} value is between 0.10 and 0.35. Lower ν_{st} values are associated with highly fractured rock masses, and higher ν_{st} values with intact rock masses.

Depending on the site conditions, in-situ testing with rock pressure meters (dilatometers) or borehole jacks may also be considered to estimate E_{st} of the rock masses as identified in Section 3.1.1.

Equivalent linear rock stiffness properties may further be adjusted based on the results of non-linear FIA as described in Section 5.1.3.

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8.54 ASCE 7-16, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers," 2016.

8.55 ACI 318-14, "Building Code Requirements for Structural Concrete, American Concrete Institute," 2014.

8.56 ICC 500, "ICC/NSSA Standard for the Design and Construction of Storm Shelters," International Code Council, 2014.

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8.58 ANSI/AISC 360-16, "Specification for Structural Steel Buildings, American Institute of Steel Construction," 2016.

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8.60 EPRI-102293: "Guidelines for Determining Design Basis Ground Motions," Electric Power Research Institute (EPRI), Palo Alto, CA, 1993.

8.61 Iowa DOT 200E-1, "Geotechnical Design: Engineering Properties of Soil and Rock," Iowa Department of Transportation (DOT) Office of Design, Chapter 200, May 2015.

8.62 Lambe, T.W. and R.V. Whitman, "Soil Mechanics," Massachusetts Institute of Technology, John Wiley & Sons, 1969.

8.63 FHWA NHI-16-072: "Geotechnical Site Characterization," U.S. Department of Transportation, 2017.

8.64 RG 1.132, "Site Characterization Investigations for Nuclear Power Plants," Revision 2, October 2003.

~~8.63~~8.65 RG 1.138, "Laboratory Investigations of Soil and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, December 2014.

SRP Review Section: 02.05.04 - Stability of Subsurface Materials and Foundations

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02.05.04-01 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Sections: TR NEDO-33914 Sections 3.1.1, 3.1.2, 3.1.3, and 4.3.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Issue:

Figure 4-2 shows the rheological model of an interface to be used in the Foundation Interface Analysis (FIA) with several parameters, such as, k_n, σ_t, τ_{max}, k_s, C, C_r, φ. These parameters determine whether the interface slides (shear failure) or dilates (tensile failure) under the imposed loads including the load from the safe shutdown earthquake (SSE). The response of both soil and rock media surrounding the Reactor Building (RB) shaft to the imposed loads significantly affects the loads imposed on the RB walls. In addition, the loads imposed on the RB wall may not be symmetric around the shaft walls, especially in the rock medium.

Section 4.3.1.2, Fault or Joint Planes or Interfaces Between Bedding Units in a Geologic Formation, states that the nonlinearity and behavior of the joints are analyzed throughout the life stages of a reactor and the same interface model would be used in modeling the joints, bedding planes, and faults in the rock mass as part of the FIA model. The properties assigned to the interface elements along a rock discontinuity are to be obtained from laboratory or field testing (Sections 3.1.1 and 3.1.2). In addition, Section 4.3.1.2 states that the parameters representing slip of the interface model may be estimated based on properties of the weakest interface materials.

It is not clear from the discussions given in Site Investigation Program (Section 3.1.1) and Laboratory Testing Program (Section 3.1.2) whether a specific program would be developed to collect the necessary samples at the site and conduct specific tests at the laboratory to determine the parameters of the FIA model, as shown in Figure 4-2, or any other model to be used to represent

the interfaces. It is also not clear how the weakest plane (interface) would be identified at a given site with its strength properties.

Request:

The staff requests GEH to identify the sample collection and testing programs that would be used to determine the parameters necessary to model the behavior of all interfaces (RB Wall/Soil, RB Wall/Rock, Soil/Rock, and Rock/Rock for joints/bedding planes), as appropriate. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-01

The sample collection and testing program discussed in Section 3.1 of NEDO-33914 will include samples of soil and rock materials that will be adjacent to the interfaces as well as samples of discontinuities that form rock/rock interfaces. Strength tests will be performed to characterize the shear strengths of the Reactor Building (RB) wall/rock and rock/rock interface. Shear strengths for other interfaces will be based on the strength properties of adjacent soil or rock materials. The strength testing program may also be used to measure interface stiffness parameters.

Figure 4-2 of NEDO-33914 provides an example model to simulate interactions at the RB wall/soil, RB wall/rock, soil/rock, and rock/rock interfaces. The model consists of two sets of elastic-perfectly plastic springs to simulate potential sliding and gapping at the interfaces. The properties for the interface model can be divided into strength properties, such as f_t, τ_{max}, c, c_r, and ϕ, and elastic stiffness properties, such as k_n and k_s. The strength properties are important because they determine the plastic deformations caused by either shear or tensile failures at the interface.

The stiffness properties determine the elastic deformations that are small compared to the plastic deformations and should have a minimal effect on the overall response of the nonlinear Foundation Interface Analysis (FIA). Most commercial software suitable for the nonlinear FIA have guidance for setting the k_s and k_n values (References 01-1 and 01-2). The values of the k_s and k_n are typically set in accordance with the guidance for the selected software. This guidance typically includes the use of interface stiffness values based on a value larger than adjacent materials, measured stiffness from laboratory strength tests, or other appropriate methods (References 01-1 and 01-2).

The strength properties control whether the shear failure (sliding) or tensile failure (gapping) occur along the interface under the proposed loading. The strength properties can be determined from direct shear (e.g., References 01-3 and 01-4) and/or triaxial (e.g., Reference 01-5) tests on recovered rock cores with natural discontinuities from the field investigation and artificial interfaces. If samples from specific rock discontinuities are needed, additional sampling focused on those rock discontinuities may be required. The lowest measured strength properties or the range of properties from the strength tests on representative rock discontinuities may be used to simulate movement along rock interfaces or evaluate the sensitivity of the results.

If the example interface model in Figure 4-2 of NEDO-33914 is not used, the site investigation and laboratory testing programs shall be modified to determine the parameters for the selected interface model.

References

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01-2. Itasca, FLAC3D Manual – Theory and Background, 2019 (updated 10/01/2021) <http://docs. itascacg.com/flac3d700/flac3d/docproject/source/flac3dhome.html?node1877>.

01-3. American Society for Testing and Materials ASTM D5607, "Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force," 2016.

01-4. U.S. Army Corps of Engineers, "Method of Test for Direct Shear Strength of Rock Core Specimens," RTH 203-80, Waterways Experiment Station, Vicksburg, MS, 1993.

01-5. U.S. Army Corps of Engineers, "Standard Method of Test for Multistage Triaxial Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements," RTH 204-80, Waterways Experiment Station, Vicksburg, MS, 1993.

Proposed Changes to NEDO-33914 Revision 0

Section 3.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to include direct shear and triaxial strength tests for natural and artificial discontinuities. Direct shear tests are added to the minimum laboratory tests required for rock materials.

Section 3.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to identify direct shear and triaxial compressive tests as laboratory tests on recovered samples of discontinuities.

Section 4.3.1.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to state that the interface parameters are from adjacent soil/rock elements or strength test on natural and artificial discontinuities developed to be consistent with the selected nonlinear FIA software and interface model. Uncertainties in these interface parameters may require sensitivity analyses that adjust the spring stiffness and shear strength using strength reduction factors. Removed text discussing the potential boundary scenarios.

Section 4.3.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to indicate that the weakest strength parameters from multiple tests on rock discontinuities may be used for interface elements and that strength reduction factors may be used to modify the interface values for strength and stiffness.

Section 8.0 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to add ASTM D5607, RTH 203-80, RTH 204-80, RG 1.132 and RG 1.138 to the list of references.

02.05.04-02 (eRAI 9859)
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LTR Application Sections: TR NEDO-33914 Sections 3.1.1, 3.1.3, 3.2.1, 4.2.2, and 5.2.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Revision 2, describes methods acceptable to the NRC staff for conducting field investigations to acquire the geological and engineering characteristics of the site and provides recommendations for developing site-specific guidance for conducting subsurface investigations.

RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, describes laboratory investigations and testing practices for determining soil and rock properties and characteristics needed for engineering analysis and design of foundations and earthworks for nuclear power plants.

Issue:

It is not clear from the discussion given in Section 3.1.3, Characterization of Rock Mass Properties, whether the geological characterization of the rock unit(s) would be adequate to represent the rock mass in the FIA analyses. Discussions on fracture network characterization of the rock mass is mostly limited to collecting 1D information through boreholes. However, rock fractures are 3D in nature and occur in sets (joint sets). Multiple sets of rock joints can be present in a rock mass creating individual rock blocks. Additionally, the rock mass may be a bedded deposit comprising of multiple rock beds. No discussion is given in Section 3.1.3 how the rock mass fracture network, which can significantly influence the rock pressure of the RB walls, would be characterized.

Additionally, Section 3.1.3, Characterization of Rock Mass Properties, discusses the use of rock mass classification systems (e.g., the Rock Mass Rating (RMR) system, the Geological Strength Index (GSI) system) to develop an estimate of the stress-strain behavior of rock (Section 4.2.2, Rock Constitutive Model) and rock mass stiffness properties (Section 5.2.1.2, Rock Mass

Equivalent Linear Properties). The RMR system specifically requires information of the rock discontinuity spacing, orientation, and conditions. The GSI system requires information on at least J_r (joint roughness number) and J_a (joint alteration number) parameters to determine the specific GSI value of the rock mass. It is not clear how these parameters would be determined based on discussion given in Section 3.1.1, Site Investigation Program. Additionally, it is not clear what inspection and verification programs would be used during the Construction Phase (Section 3.2.1, Excavation and Foundation Inspections and Testing) to verify the assumptions made about the rock mass (e.g., rock fracture network, joint strength, etc.) before the excavation commences.

Request:

The staff requests GEH to identify the plan(s) and program(s) for characterizing the rock fracture network and determining the necessary parameters for the rock mass classification system used to determine the rock mass stress-strain behavior. The staff is also requesting GEH to identify the program(s) to verify the assumptions made of the rock and soil media surrounding the RB shaft as the excavation progresses. Modify the TR, as necessary.

GEH Response to NRC Question 02.05.04-02

The program for characterizing the rock fracture network and determining the necessary parameters for the rock mass classification system at depths for engineering purposes (see the GEH Response to NRC Question 02.05.04-08) will include four primary data collection components (Reference 02-1):

- geologic mapping prior to the field investigation;
- intrusive boreholes during the field investigation;
- laboratory testing of recovered rock cores; and
- geologic mapping of the excavation during construction.

The geologic mapping, field investigation, and laboratory testing data are used to estimate parameters for design and analysis. The excavation mapping data collection is intended to confirm the rock conditions were realistically estimated for analysis and design when bedrock is at or above the basemat level of the Reactor Building (RB) shaft (Reference 02-1).

Following a review of the available geologic data, surface mapping of identified rock outcrops and surface geologic features will be completed. At a minimum the mapping is intended to:

- identify the rock structure;
- the orientation, spacing and persistence of principal joint sets; and
- characterize discontinuities, shear zones, faults, and other potentially weak planes in the bedrock units.

Based on the results of the surface mapping, the recommended field investigation plan provided in Section 3.1.1 of NEDO-33914 may be supplemented to include:

- inclined borings at orientations to better intersect the anticipated discontinuities;
- additional borings to investigate specific structural features; or
- larger diameter samples to recover natural discontinuities.

The geotechnical borings will provide recovered cores, televiewer measurements, seismic measurements, access for water-pressure tests in bedrock, and allow installation of piezometers. This data would be used to supplement the surface mapping and better characterizing the rock mass. Surface geophysical measurements between the borings will also be completed to aid in mapping variations in the bedrock units as indicated in Table 3-1 of NEDO-33914.

The laboratory investigation described in Section 3.1.2 of NEDO-33914 includes testing of recovered rock core samples to characterize the rock mass. These rock cores will be from the borings in the field investigation. The exact testing of the recovered rock cores would be based on the selected rock mass classification systems, but tests on recovered rock cores would potentially include:

- Uniaxial compressive and triaxial strength testing of intact rock cores (Reference 02-2).
- Tensile strength testing of intact rock cores and natural discontinuities (Reference 02-3).
- Direct shear and/or triaxial testing of natural or artificial discontinuities (References 02-4, 02-5, and 02-6).

As described in Section 3.2.1 of NEDO-33914, the walls and floors of the rock excavation will be mapped during and at the completion of the excavation when bedrock is present at or above the basemat level of the RB shaft. This excavation mapping is to verify the rock mass characteristics estimated during design and analysis are appropriate and will be completed in accordance with Appendices A and B of Reference 02-7. If the rock characteristics observed during the excavation do not conform to those estimated for the analysis and design, sensitivity evaluations will be performed to assess their effect on the design.

References

02-1. RG1.132, “Site Investigations for Foundations of Nuclear Power Plants,” Revision 2, October 2003.

02-2. American Society for Testing and Materials ASTM D7012, “Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures,” 2014.

02-3. American Society for Testing and Materials ASTM D2936, “Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens,” 2020.

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Proposed Changes to NEDO-33914 Revision 0

Section 3.1.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to identify the need for geologic mapping of outcrops to improve the field investigation program and the rock characterization when bedrock is encountered at depths for engineering purposes.

Table 3-1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to add “characterize rock mass and discontinuities” to test purposes for geotechnical borings and the borehole televiewer.

Section 3.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to identify the investigation locations and methods, including geologic mapping, intended to characterize the rock and rock mass parameters.

02.05.04-03 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Sections: TR NEDO-33914 Sections 3.2, 3.3, 3.4, and 4.1

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Revision 2, describes methods acceptable to the NRC staff for conducting field investigations to acquire the geological and engineering characteristics of the site and provides recommendations for developing site-specific guidance for conducting subsurface investigations.

RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, describes laboratory investigations and testing practices for determining soil and rock properties and characteristics needed for engineering analysis and design of foundations and earthworks for nuclear power plants.

Issue:

Section 4.1, Foundation Interface Analysis Model, states that a numerical model of the interfaces would be developed that examines the response of the BWRX-300 and its surrounding media due to alterations of in-situ subgrade conditions. The responses would be monitored, both through the FIA model response and field measurements. The numerical FIA model will also be calibrated using the field measurements to predict future response of the structure. It is not clear from the discussions in Sections 3.2, 3.3, and 3.4 how the predicted interface behavior would be compared against physical observations from the monitoring programs. Sections 3.2, 3.3, and 3.4 do not discuss any plan or program to monitor the shear and normal displacements along an interface, as shown in Figure 4-1.

Request:

The staff requests GEH to identify the plan(s) or program(s) to monitor the response of the BWRX-300 and its surrounding media and comparing them with predictions using the FIA model for

calibrating the numerical FIA model. Additionally, this process should verify that the structural and site responses are within the design bounds. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-03

As discussed in Section 3.4 of NEDO-33914, a field instrumentation plan will be developed to monitor deformations, groundwater pressures and soil and rock pressures during excavation, construction, loading and operation. Specific locations will be determined based on the subsurface conditions and areas identified in the design where deformation and pore pressures are anticipated along the perimeter of the Reactor Building (RB) shaft. The specific locations may be inside and outside of the RB shaft. These measurements can be benchmarked against design estimates and used to calibrate the nonlinear Foundation Interface Analysis (FIA) model by modifying select parameters.

During construction and in-service operation, the nonlinear FIA model will be calibrated based on actual properties of RB structures and the measured response of soil and rock surrounding the RB shaft. The RB structure properties can be updated based on the as-built measurements and testing results described in Section 3.2.2 of NEDO-33914. The properties of soil and rock models will be calibrated in the nonlinear FIA based on measured deformations of the soil and rock as well as measured pore water pressure from piezometers. Select inputs or parameters of the soil, rock and rock/rock interfaces associated with the difference between modeled and measured results will be modified. If a Hoek-Brown material model is used for the rock mass that is producing less deformation than measured, then an example would be modifications to Geologic Strength Index (GSI), disturbance factor (D), or intact rock modulus (E_{ri}) used to estimate the rock mass modulus (E_{st}) or direct modification of E_{st} to better match the measured deformations.

The earth pressure from the calibrated nonlinear FIA will be compared with that from the initial nonlinear FIA and the design Soil-Structure Interaction (SSI) analysis as described in Section 5.1.3 of NEDO-33914 to ensure the pressures are within the design bound with an adequate margin. The adequacy of the margins is evaluated based on the levels of uncertainty in the subgrade conditions. The design SSI analysis is the 1-g static SASSI (a system for analyses of soil structure interaction) analysis described in Section 5.1 of NEDO-33914.

Proposed Changes to NEDO-33914 Revision 0

Section 3.4 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to indicate that the specific locations of sensor may be inside and outside of the RB shaft.

Section 4.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to include a description of the calibration process using parameters for the selected soil and rock constitutive models.

Section 5.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to clarify that the one-step approach is implemented using a linear elastic SASSI analysis approach.

02.05.04-04 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

In Section 5.1.2, Soil-Structure Interaction Modeling Assumptions, the rock mass is assumed to be continuous, and the presence of cavities, fracture zones, joints, bedding planes, discontinuities, and other weak zones is neglected. It is not clear from the discussion whether their effects on the rock mass properties (e.g., rock mass modulus, strength) would be incorporated through equivalent rock mass properties so that the calculated loads on the RB walls are realistic. It is also not clear whether an isotropic assumption of the equivalent material properties would be made. Rock fractures have specific orientations in the 3D space and make the rock mass properties anisotropic.

Request:

The staff requests GEH to provide a discussion in the TR how the effects of rock fractures etc. would be incorporated in the SSI modeling.

GEH Response to NRC Question 02.05.04-04

A continuous model is anticipated to be used for the nonlinear Foundation Interface Analysis (FIA), as described in Section 4.0 of NEDO-33914, to simulate the Reactor Building (RB) structure and subgrade conditions for the excavation, construction, loading, startup and operation stages. Mohr-Coulomb or Hoek-Brown material models will typically be used to represent massive rock units and jointed or fractured rock units that approximate a continuum. Highly anisotropic features like bedding planes and select discontinuities may be incorporated into the continuous model using rock/rock interfaces described in the GEH response to NRC Question 02.05.04-01. The use of discontinuous models is not anticipated for most sites because the excavation would be reinforced for unstable rock masses during excavation and construction, and the RB shaft would be present after construction. A continuous model is typically appropriate because the rock reinforcement and the RB structure would constrain the rock mass and limit the deformations.

Possible fracture zones, joints, bedding planes, discontinuities and cavities in the rock are not explicitly included in the design Soil-Structure Interaction (SSI) analyses providing demands for the RB structures that are described in Section 5.0 of NEDO-33914. The design demands are obtained from SASSI analyses based on the principles of continuum mechanics using isotropic and linear elastic constitutive models for all materials, including the rock materials.

The design of deeply embedded RB structure must consider different environmental and plant operating loads that are combined in different design load combinations per the governing design codes. The superposition principle, which is applicable only for linear elastic analyses, is essential for the design because it allows the results of the different dynamic, static and thermal stress analyses to be combined in different load combinations. The linear elastic assumption also allows the one-step design SSI analyses in Section 5.1 of NEDO-33914 to be performed on a refined RB structural model with many degrees of freedom and eliminates the need for defining initial conditions for each design load combination to calculate the structural design demands.

Sections 5.1.3 of NEDO-33914 describes the approach used to ensure the RB design bounds the subgrade conditions with adequate margins through comparisons of the results of:

- the linear-elastic 1-g SASSI analysis that provides static earth pressure demands used for the design of RB structure, and
- the nonlinear FIA that are performed following the guidelines in Section 4.0 of NEDO-33914.

The nonlinear FIA provides more realistic estimates of the earth pressures on the RB shaft by using models that better represent the response of fracture zones, joints, bedding planes, discontinuities or cavities in the rock.

Additional design analyses may be performed where earth pressure loads are applied to the below grade exterior walls of the refined RB structural model to account for:

- the effects on the RB design of anisotropic or heterogenous rock responses that cannot be directly modeled by the isotropic-elastic SASSI model; or

- potential pressures from unstable blocks of rock mass.

The magnitude and distribution of these additional earth pressure loads are determined from the results of nonlinear FIA or other estimates of earth pressures from unstable blocks of rock mass calculated as described in the GEH response to NRC Question 02.05.04-05 and in Sections 5.1.3 and 5.1.4 of NEDO-33914. The structural design demands obtained from the additional earth pressure analysis are combined with the results of the one-step design SSI analysis, as described in Section 5.1 of NEDO-33914, to ensure the RB structural design adequately addresses the effects of anisotropic and heterogenous rock behavior and accounts for potential unstable rock mass loads.

Proposed Changes to NEDO-33914 Revision 0

Assumption (1) in Section 5.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to clarify that the properties of subgrade materials for the design SSI analyses performed using the SASSI methodology are assumed isotropic.

Section 5.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to describe the approach used to account in the design for the effects of anisotropic or heterogenous rock response including potential pressures from unstable blocks of rock mass.

02.05.04-05 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

Section 5.1.2, Soil-Structure Instruction Modeling Assumptions, states that "Strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness will frequently be self-supporting even if some reinforcement is required to ensure a safe excavation." It is not clear what is meant by disadvantageous fracture zones, joints, bedding planes, discontinuities, and other zones of weakness, and how they will be identified at a site.

It is further stated that "Joints and other weak planes may create isolated blocks that are unstable; however, these blocks are not typically large relative to the area of the structure and would be unlikely to produce significant loads on the exterior of the structure compared to other loads (e.g., hydrostatic). These blocks would also not be able to create a cascading failure once the structure is in place." It is not clear what are the basis for the assumption that unstable blocks would be isolated. It is also not clear why the unstable blocks would not produce significant loads on the RB structure. The unstable blocks could impose concentrated load(s) with significantly higher magnitude than the hydrostatic load on the RB walls (e.g., the scenario shown in Figure 5-1). It is also not clear from the discussion how the design of the RB structure would account for such large rock mass failure.

Request:

The staff requests GEH to provide an approach to identify the disadvantageous fracture zones, joints, bedding planes, discontinuities, and other zones of weakness at a site. The staff also requests GEH to provide rationale why the unstable blocks would not produce significant loads on the RB structure and explain how the design of the RB structure would account for such load. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-05

The text describing the smaller loads on the Reactor Building (RB) structures from isolated blocks will be removed from Section 5.1.2 of NEDO-33914. The text is misleading because the load from blocks and wedges would be estimated. Blocks and wedges smaller than the maximum size would likely not be as significant on the RB structures, but these loads could be estimated.

The general approach for identifying fracture zones, joints, bedding planes, discontinuities, and other zones of weakness at a site are provided in the GEH response to NRC Question 02.05.04-02 concerning characterization of the rock fracture network. As stated in the GEH response to NRC Question 02.05.04-02, characterization of the rock fracture network may include additional or inclined borings to investigate specific discontinuities, structural features, and recover rock cores with natural discontinuities.

Discontinuities identified through mapping, field investigation and laboratory testing may form unstable blocks or wedges that could potentially move towards the excavation without reinforcement (Reference 05-1). Therefore, identification of these blocks and wedges is important for identifying potentially unstable excavations that require reinforcement. For the BWRX-300, a stable excavation would either have no unstable blocks and wedges or would be stabilized by reinforcement. A stabilized excavation would potentially result in earth pressures on the RB shaft after degradation of the reinforcement as discussed in the GEH response to NRC Question 02.05.04-06 and in Section 5.1.2 of NEDO-33914.

Identification of unstable blocks and wedges will typically use a graphical method or computer program to implement block theory (References 05-1 and 05-2). Block theory is used to identify potential blocks and wedges that may form at the perimeter of the excavation (Reference 05-2). This process evaluates the orientation data of discontinuities from mapping and borehole measurements to determine the number of principal joint sets in the bedrock units. The average orientation as well as the variation in the average orientation (e.g., Fisher constant) is determined for the strike and dip of each set (Reference 05-3). A graphical method or computer program can then determine if the different joint sets form finite and removable blocks that are potentially unstable (References 05-1 and 05-2). The maximum unstable block (key block) can be determined once the excavation dimensions are considered, and the maximum block size can be further refined when data on the spacing of the discontinuities is incorporated (Reference 05-2).

Estimates of earth pressures on the RB shaft from the maximum unstable block size can be obtained from:

- Nonlinear Foundation Interface Analysis (FIA) that includes rock/rock discontinuities represented by interface models as described in Section 4.3.1.2 of NEDO-33914 and in the GEH response to NRC Questions 02.05.04-01 and 02.05.04-04; and/or
- Force equilibrium analyses described in Section 5.1.4 of NEDO-33914.

The primary focus in estimating the earth pressure will be on the maximum block sizes and unstable blocks that could produce larger loads. If the load from specific blocks or wedges is large, alternative mitigation measures may be considered, such as overexcavating and backfilling or the use of degradation resistant reinforcement.

References

05-1. Hoek, E., Practical Rock Engineering, RocScience < https://www.rocscience.com/assets/resources/learning/hoek/Practical-Rock-Engineering-Full-Text.pdf >, 2007.

05-2. Goodman, R.E., and Shi, G., Block Theory and its Application to Rock Engineering, Prentice-Hall, Inc, 1985.

05-3. Brady, B.H.G., and Brown, E.T., Rock Mechanics for Underground Mining, 3rd Edition, Kluwer Academic Publishers, 2004.

Proposed Changes to NEDO-33914 Revision 0

Section 5.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to remove the text stating isolated unstable blocks do not produce significant loads. The text is modified to indicate an appropriate method (e.g., block theory) should be used to identify potentially unstable blocks and wedges and that the nonlinear FIA may be used to estimate the potential loads of unstable blocks and wedges.

Section 5.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to include other mitigation methods like overexcavation and backfilling when the potential load from a block or wedge is large.

Section 8.0 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to include Goodman and Shi (1985) as a reference.

02.05.04-06 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.2

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

Section 5.1.2, Soil-Structure Instruction Modeling Assumptions, assumes that the rock is self-supporting. It is not clear whether the BWRX-300 reactor system cannot be installed in a rock mass that is not self-supporting, e.g., rock mass with poor rock quality.

Request:

The staff requests GEH to clarify whether a site requiring significant permanent support system(s) to keep the surrounding media stable would be unsuitable for siting a BWRX-300 reactor. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-06

The BWRX-300 can be deployed at soil sites and sites having rock masses that require support during the excavation and construction of the deeply embedded Reactor Building (RB) shaft. The as-built site-specific subgrade conditions must ensure the stability of the BWRX-300 power block foundations by conforming to the regulatory requirements of NUREG 0800 Standard Review Plan

(SRP) 2.5.4, “Stability of Subsurface Materials and Foundations” (Reference 06-1). The stability of the surface mounted foundations of the surrounding Radwaste Building (RwB), Control Building (CB) and Turbine Building (TB) is crucial for the stability and structural integrity of the deeply embedded RB. Therefore, the SRP 2.5.4 requirements shall be satisfied not only for the Seismic Category 1 RB foundation but also for the surrounding surface mounted foundations.

Nonlinear Foundation Interface Analysis (FIA) are performed, as described in Section 4.0 of NEDO-33914, to evaluate the suitability of the site for deployment of BWRX-300 by assessing the stability of the supporting media, soil and/or rock, and RB, CB, TB and RwB foundations per SRP 2.5.4. The nonlinear FIA are performed for different life stages starting from the site characterization, excavation, construction, loading to start-up and operation. The results of the analysis of each stage are used to establish the initial conditions for the subsequent stage. The excavation support elements, such as rock reinforcement and soil support are included in the FIA model used for the analyses of the excavation, construction and loading stages.

The GEH response to NRC Question 02.05.04-05 describes the approach used to identify potentially unstable blocks and wedges in the rock mass that may require reinforcement. When a rock mass requires reinforcement, it is not considered stable over the life of the BWRX-300. The excavation support elements are considered temporary and are only included in the nonlinear FIA for the excavation, construction, and loading stages. The excavation support elements are removed from the nonlinear FIA for the startup and operation stage to calculate the earth pressures on the RB shaft. These earth pressure results can be compared with:

- the earth pressure loads considered for the conceptual generic design of the RB structure to obtain a general first-hand assessment of the suitability of the BWRX-300 generic design for the site-specific subgrade conditions; and,
- the earth pressure results of the site-specific design SSI analyses, as described in Section 5.1.3 of NEDO-33914, for the final assessment of the RB structural design suitability for the site.

For sites where the subsurface conditions may result in large loads to the RB shaft, the BWRX-300 may still be deployed with additional mitigation methods beyond a more robust design of the RB structure. As described in the GEH response to NRC Question 02.05.04-05, these mitigation methods to reduce the loads on the RB shaft may include overexcavation and backfilling, installation of degradation resistant rock reinforcement, or other methods.

Reference

06-1. NUREG-0800 Standard Review Plan (SRP) 2.5.4, “Stability of Subsurface Materials and Foundations,” Revision 5 (Reference 06-1).

Proposed Changes to NEDO-33914 Revision 0

Assumption (4) in Section 5.1.2 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to clarify that the design analyses can only neglect the rock pressures due to the weight of rock masses that are not required to be laterally supported during the excavation and construction.

02.05.04-07 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 4.3.3

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Issue:

Section 4.3.3, Fluid-Soil Interaction, states that the 3D model of BWRX-300 may have hydraulic interface(s) to simulate the effects of pore water during excavation, construction, loading, and operation phases of the reactor. In rock, flow through the rock fracture network can be the dominant flow mechanism. It is not clear what approaches would be taken to deal with fracture flow if it is present.

Request:

The staff requests GEH to clarify the approach to account for fracture flow. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-07

Explicit simulation of fractured flow is not anticipated for the model outlined in Section 4.3.3 of NEDO-33914. The model of potential groundwater flow would be used to evaluate changes in the groundwater level from high and low water tables or dewatering activities. Therefore, simulating the rock as a continuum is considered adequate for most sites. The fractured flow of groundwater in rock will likely be affected or reduced by grouting and/or dewatering near the excavation for the Reactor Building (RB) shaft by construction activities as noted in Section 3.1.3 of NEDO-33914. Monitoring of the water pressure in the rock mass will be completed as part of the field instrumentation plan in Section 3.4 of NEDO-33914.

The influence of groundwater will be addressed following the recommendation of the selected rock mass characterization method. The Rock Mass Rating (RMR) system uses a groundwater parameter and rating value to account for the groundwater in the fractured rock mass (Reference 07-1). The RMR groundwater rating is based on (Reference 07-1):

- inflow per 10 meters of tunnel length;
- the stress ratio of water pressure in the joints to the major principal stress in the rock mass; and,
- general conditions (dry, damp, wet, dripping, flowing).

At most sites, the inflow measurement will be impractical until the shaft excavation begins, and when the shaft is present the inflow value may be altered by grouting or other modifications for construction groundwater control. Measurements of the groundwater pressure using piezometers installed as part of the field investigation and monitoring program may be used to measure the water pressure in discontinuities and the rock mass. These water pressure measurements could then be used with the in-situ stress measurements in the rock mass, described in the GEH response to NRC Question 02.05.04-08 to assign the groundwater rating for RMR based on the stress ratio. However, the RMR groundwater rating may also be based on the qualitative general conditions description that would include observations from the field investigation and conservative estimates of the appropriate rating (Reference 07-1).

The Geological Strength Index (GSI) characterization of rock mass incorporates the potential deterioration of shear strength along rock discontinuities in wet conditions by recommending a shift to lower values in rocks with fair to very poor discontinuity surface conditions (Reference 07-2). Water pressure does not modify the GSI value; however, it is included by effective stress analysis using an appropriate material model (e.g., Hoek-Brown, Mohr-Coulomb).

Other rock mass ratings for RMR and GSI (e.g., uniaxial compressive strength; rock quality designation; rock structure; and, spacing, condition and orientation of discontinuities) would be based on the fracture characterization and rock data described in the GEH response to NRC Question 02.05.04-02.

References

07-1. Brady, B.H.G., and Brown, E.T., Rock Mechanics for Underground Mining, 3rd Edition, Kluwer Academic Publishers, 2004.

07-2. Hoek, E., Brown, E.T., “The Hoek–Brown failure criterion and GSI – 2018 edition,” Journal of Rock Mechanics and Geotechnical Engineering, 11(3), pp. 445–463, 2019.

Proposed Changes to NEDO-33914 Revision 0

Section 3.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to state that the groundwater conditions shall be included according to the selected method in evaluating rock mass characterization.

02.05.04-08 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 3.1.1

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Issue:

The TR does not state whether there would be a program to measure the in-situ stress field at the site. In Section 3.1.1, Site Investigation Program, the maximum required drilling depth is set at 120 m because the expected change in stresses due to excavation of the shaft would be less than 10% from the original in-situ stress field. It is not clear how this can be set without knowing the in-situ stress field. The stress distribution around the RB shaft could be quite different if horizontal stresses are larger than the vertical stress affecting the loads on the RB shaft walls.

Request:

The staff requests GEH to clarify whether there will be process to measure the in-situ state of stresses at the site and incorporate the stress field in the analyses. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-08

There will be a process to identify the in-situ state of stresses at the site and, when necessary, measure and incorporate the in-situ state of stress into analyses. Identification of the in-situ state of stress shall begin with reviewing the state of stress in the crust as part of evaluating the tectonic framework and unrelieved stresses in bedrock near the site.

A review of regional and/or local references that evaluate the current state of stress in the crust and the potential for horizontal stresses from tectonic activity, residual strains, or topographic conditions shall be used to assess the likelihood for increased horizontal stress in the bedrock (NUREG/CR-5738, Reference 08-1). Based on the results of this review, in-situ tests will be

considered to make site-specific measurements of the in-situ state of stress in bedrock formations as part of the geotechnical borings and televiewer tests shown in Table 3-1 of NEDO-33914.

Overcoring and hydraulic fracturing are identified in Table 3-1 of NEDO-33914 as potential methods for estimating the in-situ stress; however, other methods including the identification of the principal stress directions and relative magnitudes from those identified in RG 1.132 (Reference 08-2) and breakouts recorded by borehole televiewer may be used (e.g., Reference 08-3). The appropriate tests for measurement of in-situ stress will be selected based on the likelihood for increased horizontal stresses and the type of subsurface materials at the site. NEDO-33914 does not list all tests for measuring in-situ stress because the selection of the appropriate tests is specific to each site.

When the in-situ state of stress is different than lithostatic in bedrock units, this difference will be incorporated into the initial conditions of the nonlinear Foundation Interface Analysis (FIA) model. The nonlinear FIA model and the potential key blocks described in the GEH response to NRC Question 02.05.04-05 may then be used to evaluate potential loads from an excavation with the in-situ stress field. These loads will be included in the design as described in the GEH response to NRC Question 02.05.04-04.

Consistent with Appendix D of RG 1.132 (Reference 08-2), the maximum required drilling depth for engineering purposes (d_{max}) of 120 m is based on a change in the vertical stress.

References

08-1. U.S. Nuclear Regulatory Commission, “Field Investigations for Foundations of Nuclear Power Facilities,” NUREG/CR-5738, ADAMS Accession No. ML003726925, November 1999.

08-2. RG 1.132, “Site Characterization Investigations for Nuclear Power Plants,” Revision 2, October 2003.

08-3. Science Applications International Corporation (SAIC), “Final Report – In Situ Stress Measurements in the NPR Hole,” Report Submitted to Westinghouse Savannah River Company, ADAMS Accession No. ML013190312, July 1992.

Proposed Changes to NEDO-33914 Revision 0

Section 3.1.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to define d_{max} based on the vertical stress and as the depth for engineering purposes.

Table 3-1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to correct “in-site stress” to “in-situ stress” for the test purpose of geotechnical borings and borehole televiewer.

Section 3.1.3 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to include a discussion on reviewing the current knowledge of the state of stress in the bedrock and, as needed, measurements of the in-situ stress state.

Section 4.3.4.1 of NEDO-33914 is being revised as shown in the Enclosure 2 markups to state the initial stress conditions include the influence of groundwater and measured horizontal stresses for the nonlinear FIA.

02.05.04-09 (eRAI 9859)
Date of eRAI Issue: 07/30/2021
LTR Application Section: TR NEDO-33914 Section 5.1.4

Requirements:

General Design Criterion (GDC) 2 requires that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety function. GDC 2 also specifies that the design bases for these SSCs shall reflect the importance of the safety functions to be performed.

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

Standard Review Plan (SRP) NUREG-0800, Section 2.5.4, provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.1, provides regulatory guidance for the development of site design ground motion acceleration response spectra and time histories.

Standard Review Plan (SRP) NUREG-0800, Section 3.7.2, states for the seismic design of nuclear power plants, it is customary to specify earthquake design ground motions that are exerted on the plant structures and used in soil-structure interaction (SSI) analyses.

Issue:

Section 5.1.4, Probabilistic Earth Pressure Analyses, presents an approach as an example to account for the uncertainties in the estimated value of the load on the RB shaft walls in a soil medium. It is not clear whether similar approaches would be used to account for uncertainties in at least important parameters significant to the reactor design; for example, the estimated rock mass modulus estimated using various empirical equations from different measured and inferred parameters (indirect estimation).

Request:

The staff requests GEH to clarify whether there will be a plan to account for the uncertainties in other site-related parameters with potential to significantly affect the reactor design. Modify the TR as necessary.

GEH Response to NRC Question 02.05.04-09

The design accounts for uncertainties in all site-related parameters that can potentially have a significant effect on the stability of the BWRX-300. The uncertainties in the site-related parameters can be addressed deterministically by considering conservative values, performing sensitivity analyses for multiple values, or probabilistically based on results of statistical analyses.

Conservative site-related design parameters are developed, as described in Section 5.2 of NEDO-33914, considering the natural randomness of subgrade materials and errors in measurement of the subgrade material properties. Uncertainties related to the methods or empirical relationships used for development of site-related design parameters are also addressed by considering different models. For example, uncertainties in the estimated rock mass modulus may consider different models as the ones shown in Equations 5-17 to 5-22 of NEDO-33914. The uncertainties in the measured and estimated parameters used in these models, such as Rock Mass Rating (RMR), Geological Strength Index (GSI), disturbance factor, modulus ratio, Uniaxial Compressive (UC) strength, intact rock modulus, are also considered either deterministically or probabilistically.

Using the same approach implemented for probabilistic analyses in Section 5.1.4 of NEDO-33914, the statistical analyses performed to develop the site parameters may consider:

- probabilistic distributions, usually normal or log-normal, to address the aleatory variability of the measured parameters; and
- discrete probabilistic distribution combined using appropriate weight factors to address epistemic uncertainties related to the use of different measuring techniques or methods for development of the site parameters.

When the results of the deterministic analyses show the site-related parameters are sensitive to the input values or indicate limited margin in the design, a probabilistic analysis will be completed to demonstrate that the design adequately addresses uncertainties in the site-related parameters with potential to significantly affect the Reactor Building (RB) design. Section 5.1.4 of NEDO-33914 presents a probabilistic approach used to ensure the design adequately addresses the uncertainty in the estimated soil and rock pressures that are the most important site-related load affecting the stability of the deeply embedded RB structure. However, the same probabilistic approach can be used to address uncertainties in other important site-related design parameters.

Proposed Changes to NEDO-33914 Revision 0

Section 5.1.4 of NEDO-33914 is being revised as shown in the Enclosure 2 mark-ups to explain that the probabilistic earth pressure evaluations also consider uncertainties in the methods or empirical relationships used for development of site related parameters following an approach similar to the approach used to account for modelling uncertainties.

Table 5-1 of NEDO-33914 is being revised as shown in the Enclosure 2 mark-ups to indicate that the models for probabilistic analyses consider the natural and measurement uncertainties in rock mass properties.

3.1.1 Site Investigation Program

Figure 3-1 represents a preliminary layout of the BWRX-300 footprint and facilities with the deeply embedded RB being the only SC-I structure in the BWRX-300 plant. It is common practice to perform borings and tests below the footprint of the SC-I facilities and to deeper depths than the basemat (RG 1.132, Reference 8.64). The excavation approach minimizes the use of engineered backfill materials as well as the deployment depth of the BWRX-300 RB and requires a subsurface investigation that covers areas beyond its foundation perimeter.

When bedrock units are anticipated to be encountered at depths for engineering purposes, geologic mapping of outcrops should be completed prior to finalizing the number, orientations, and locations of the field investigation borings and tests. This geologic mapping is intended to characterize the anticipated rock mass, discontinuities and to allow for modification of the field investigation to collect appropriate data near the RB shaft.

The diameter of the RB SC-I footprint is relatively small when compared to footprints of typical conventional nuclear plants. The characterization of a small portion of the subsurface environment would be insufficient to adequately characterize the variations and uncertainties in the site subsurface conditions and provide inputs for the Approach 3 probabilistic SRA described in Section 5.2.2. Tests, such as seismic refraction or reflection studies that are useful to map bedrock or detect potential voids become meaningful and possible only when covering greater areas. Measurements of shear-wave velocities (V_S) and compression-wave velocities (V_P) are not sufficient to characterize lateral variability if these are made just a few meters apart.

In order to address the specific requirements of the BWRX-300 RB design, the subsurface site investigations are performed following the guidelines of RG 1.132 for SC-I type site investigations considering the combined footprint areas of the RB SC-I foundation and the adjacent TB, CB and RwB foundations. The extended area considered by the BWRX-300 subsurface site investigation ensures an adequate characterization of the subsurface conditions under the TB, RwB and CB foundations and resulting surcharge loads, which are important for the design of the deeply embedded RB structure and seismic design of RB SC-I SSCs.

Appendix D of RG 1.132, Spacing and Depth of Subsurface Explorations for Safety-Related Foundations, specifies the need for at least one boring underneath each projected safety-related structure or 1 boring for each 900 m^2. The footprint of the main containment shaft and the above ground surrounding structures is about 1 Ha (10,000 m^2). This implies that at least 10 borings would be required for the site investigation. RG 1.132 indicates that the boring depth should go past "the maximum required depth for engineering purposes." If bedrock is encountered, then the boring should penetrate past zones of weakness that could affect foundation performance and extend at least 6 m into sound rock. For the BWRX-300, the maximum required depth for engineering purposes d_{max} is set at approximately 120 m, a depth that is the greater than the following:

a) The depth of the shaft plus twice the diameter of the shaft, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition, and

b) Twice the width of the plant's footprint, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition.

Table 3-1: Site Investigation for the BWRX-300

Test Type		Test Purpose	Number of Tests [1]
1	Geotechnical borings	– Measure Standard Penetration (SPT) – Measure Cone Penetration Resistance – Sample soils and rock for visual classification and laboratory testing – Rock Quality Designation (RQD) – Characterize rock mass and discontinuities – Perform pressuremeter tests on weak to moderately soft rock portions to have data parameter for estimation of elastic moduli – Measure in--situe stress (overcoring, hydraulic fracturing)	– 3 borings at perimeter and center of containment down to 120 m – 2 borings at perimeter of containment down to a depth of 60 m – About 18 additional borings designed to cover the footprint of the main facilities, meet the regulatory guidance, and characterize the subsurface as a unit. (see Figure 4-1)
2	Wells	– Groundwater characterization (pump and slug tests, baseline groundwater quality) – Characterize groundwater flow direction and quantify hydraulic gradients	– 9 wells at the center and edge of containment to anticipated depth of 60 m – 4 wells down to a depth of 60 m covering the footprint of the facility
3	Geophysical boring	– Measure V_P and V_S with at least two methods: seismic downhole survey, crosshole, and/or and PS Log suspension survey.	– One boring down to 120 m at center – 4 borings at perimeter of containment down – 4 borings located a distance apart from RB to allow for wider cross sections and correlations to refraction or reflection surveys
4	Refraction Survey	– For sites in which a bedrock horizon is identified by the boring program, perform seismic refraction to obtain a three-dimensional mapping of the bedrock horizon and the thickness of weathered layers	– One grid of surveys covering the footprint extension of the facility
5	Seismic reflection survey	– Identify if voids, sinkholes, karst, or faults are present beneath the footprint of the facilities	– Three longitudinal and two to three transverse reflection sections
6	Borehole Televiewer (Optical/Acoustic)	– Observe rock surface directly, subsurface lithology and structural features such as fractures, fracture infillings, foliation, and bedding planes. – Measure orientation and spacing of rock discontinuities – Packer water-pressure tests in rock – Measure in--situe stress (overcoring, hydraulic fracturing)	– Relevant for rock conditions, over which boring recovery and RQD allow for an open borehole. – The proposed 8 televiewer locations will support a better characterization of the rock mass and as a substitute for potential inspection limitations due to the construction process.

requires a reliable set of data from laboratory tests for developing geotechnical inputs characterizing the properties of each subgrade material present at the site.

A laboratory testing program is implemented that depends on the site-specific subsurface conditions, the specific analysis requirements, and the need for sufficient data to adequately characterize variations in subsurface material properties. A sufficient number of laboratory tests are performed to minimize the uncertainties in the design related to these geotechnical input parameters by providing reliable estimates for the statistical parameters (mean and standard deviation values) of the measured material properties. The systematic (bias) errors are minimized by a carefully executed equipment calibration and sample management programs. Estimates of measuring bias are developed based on comparisons of measurements of physical parameters obtained from different types of subsurface material property tests.

Testing to estimate strength parameters for appropriate rock discontinuities in bedrock units should be completed using appropriate methods that may include direct shear test (References 8.66, 8.67), triaxial strength tests (Reference 8.68), and appropriate methods identified in RG 1.138 (Reference 8.65). This testing shall determine the strength parameters (e.g., peak friction angle, residual friction angle, and apparent cohesion) of discontinuities and similar weak planes in rock. Testing of artificial interfaces may be completed to determine the strength properties at the interfaces with the RB structures.

At a minimum, the laboratory tests of soil materials include:

- Index testing (classification, weight, plasticity, grain size)
- Strength testing (shear tests, triaxial tests)
- Deformability tests (triaxial tests, consolidation tests)
- Permeability
- Chemical testing (chlorides, sulfates, pH, Resistivity)
- Dynamic tests (Resonant Column Torsional Shear (RCTS), cyclic triaxial)

The minimum laboratory tests required to develop properties for rock materials include:

- Uniaxial Compressive (UC) strength,
- Triaxial compressive strength and elastic moduli,
- Direct shear tests,
- Petrography,
- Dynamic tests (sonic pulse wave velocity, Free-Free Resonant Column velocity tests)

Other tests, such as the expansion, creep, mineralogy, erodibility, durability, X-ray diffraction tests may be performed on an as-needed basis.

3.1.3 Characterization of Rock Mass Properties

The properties of rock are characterized based on the information collected from the site investigation and laboratory testing programs described in Sections 3.1.1 and 3.1.2. Rock joints, bedding planes, discontinuities fracture and other weak zones are evaluated to determine:

- the type of temporary excavation support and improvements required during construction;
- groundwater conditions and required seepage control measures; and
- possible effects on the rock pressure loads on the RB shaft.

The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones may affect methods used to excavate rock for construction of the shaft. Methods that are used to compensate for these weak zones include:

- over-excavation and backfilling;
- internal structural support;
- spot or pattern rock reinforcement (i.e., rock bolts or anchors); and
- surface treatments (i.e., mesh, straps, shotcrete).

Additionally, the existing groundwater conditions and the potential control of seepage through cavities, fracture zones, joints, bedding planes, and discontinuities is considered. Seepage control may include slurry walls, grouting prior to excavation, grouting during the excavation, freezing, drains, dewatering wells, sumps and other methods. The existing groundwater conditions and appropriate modifications to the rock mass classification, consistent with the selected method, shall be determined as part of the Site Investigation Program in Sections 3.1.1.

The in-situ state of stress in the bedrock shall be evaluated. This process shall include reviewing the state of stress in the crust as part of evaluating the tectonic framework and unrelieved stresses in bedrock near the site. A review of regional and/or local references that evaluate the current state of stress in the crust and the potential for horizontal stresses from tectonic activity, residual strains, or topographic conditions shall be used to assess the likelihood for increased horizontal stress in the bedrock. Based on the results of this review, in-situ tests like those shown in Table 3-1 may be considered to make site-specific measurements of the in-situ state of stress in bedrock formations as part of the geotechnical borings and borehole televiewer tests. All potential and/or appropriate tests for measuring in-situ stress are not identified in this document since the appropriate tests will be specific to each site.

Discontinuities and other zones of weakness within the rock mass may also control the stability of individual blocks or the rock mass when the orientation is disadvantageous and/or the spacing of discontinuities is sufficiently dense. The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. These discontinuities or weak zones may form a system of blocks or wedges where strength within the individual blocks is high, but strength along the weak zones between the blocks is highly anisotropic.

To adequately assess and consider weak zones in rock masses, RG 1.132 and NUREG/CR-5738 (Reference 8.2) provide guidance on geologic mapping, logging and characterizing rock materials.

Geologic mapping and geotechnical borings described in Section 3.1.1 are used to characterize the intact rock, rock discontinuities, and the rock mass. Frequently, optical and acoustic televiewers (OTV/ATV) are used in conjunction with geologic mapping and oriented or classical rock coring methods to map the depths, orientations, aperture, and other characteristics of the discontinuities. The type of information and testing required for the rock mass will depend on the specific subgrade conditions as well as the rock mass classification selected for the site. When other data or geologic mapping indicates near vertical discontinuities may be present, ~~I~~inclined borings may be used to properly characterize the orientation and strength of near vertical discontinuities.

Empirical engineering and geo-mechanical rock mass classifications, such as the Rock Quality Designation (RQD) index, the Rock Tunneling Quality (Q) index, the 1976 and 1989 versions of the Rock Mass Rating (RMR) system, and the Geologic Strength Index (GSI), are used to quantitatively characterize the geologic and engineering parameters of rock masses (FHWA, 2009). These classifications often consider a variety of parameter ratings that are assigned based on the observations and measurements from characterized rock mass and may incorporate the proposed excavation techniques. Frequently, a range of parameter ratings are considered because a range of rock mass characteristics are encountered during subsurface characterization and multiple classifications systems may be considered to incorporate uncertainty in the parameter estimates.

Estimates of RQD may be made following NUREG/CR-5738 (Reference 8.2) on recovered rock cores and confirmed using OTV/ATV data or estimated from mapped or scanned surfaces based on the average number of discontinuities or volumetric joint count (Hoek et al. 2013, Reference 8.10).

RMR may be estimated following the parameters and ratings established by Bieniawski (1976, 1989, Reference 8.11). In order to use the RMR system, a rock mass is divided into different structural units defined by changes in rock type or major changes within a rock type, such as faults, fracture zones, or the spacing of discontinuities that may cause a change in the rock mass behavior. The RMR then considers semi-quantitative parameters for each structural region, which include the strength of the intact rock, RQD, the spacing of discontinuities, the condition of the discontinuities, the groundwater conditions, and the orientation of the discontinuities. Even though GSI is now commonly used directly without an estimate based on RMR, RMR is retained because previous studies have indicated better estimates using RMR for the rock mass deformation modulus of moderate to strong rock masses (Galera et al., 2007, Reference 8.12).

GSI may be estimated using qualitative charts relating the structure of the rock to the surface condition of joints for different types of rock masses (e.g., Hoek and Brown, 2018, Reference 8.13). Originally, the GSI system was developed for rock masses where block sliding and rotation was the primary means of failure without failure of the intact rock blocks, but has been extended to additional charts for other types of rock masses and geologic environments (Hoek and Brown, 2018, Reference 8.13). An appropriate GSI chart must be selected for the project site.

GSI may also be estimated semi-quantitatively for rock masses where block sliding, and rotation is the primary means of failure. This semi-quantitative method was developed for use when a qualified and experienced geologist or engineering geologist does not observe the rock mass and is recommended to supplement and not replace the qualitative estimates by a qualified and experienced professional. The quantitative input includes the RQD and the joint condition

($JCond_{89}$). Similar to the GSI, the $JCond_{89}$ value is based on a qualitative evaluation of the discontinuity surface and other features, including persistence, aperture, roughness, infilling, and weathering (Hoek et al., 2013, Reference 8.14). Alternatively, the $JCond_{89}$ may be estimated from a reduced set of estimates known as the joint roughness number (Jr) and joint alteration number (Ja) following Hoek et al. (2013, Reference 8.14). The semi-quantitative relationships for GSI and $JCond_{89}$ from Hoek et al. (2013) are provided below:

$$\mathrm{GSI} = 1.5\mathrm{JCond}_{89} + \frac{\mathrm{RQD}}{2} \tag{3-1}$$

where: $$\mathrm{JCond}_{89} = 35\frac{\left(\frac{\mathrm{Jr}}{\mathrm{Ja}}\right)}{\left(1+\frac{\mathrm{Jr}}{\mathrm{Ja}}\right)}$$

As described in RG 1.132, characterization of the shear strength for planar discontinuities, such as bedding planes, faults, fracture zones, joints, and shear zones typically include laboratory testing of subsurface discontinuities recovered from samples (e.g., direct shear and triaxial compressive strength tests) or, less commonly, in-situ tests of the discontinuities under specific loading conditions. Because the most common method is testing recovered subsurface samples, empirical corrections are required for surface roughness, intact surface strength, and the scale of the tested sample (e.g., Barton-Bandis criterion).

When the rock discontinuities are filled with another material, the shear strength may decrease or increase depending on the type of infill material. Testing of the infill material is required when there is a significant thickness of weaker material that may control the strength of the discontinuity. When a nonlinear relationship between shear strength and normal stress (e.g., Barton-Bandis criterion) is not desired, the equivalent friction angle and cohesion may be determined from the tangent to the nonlinear relationship for the shear strength of planar discontinuities.

Cavities in the rock mass from karst or dissolution may decrease the effective rock mass modulus and create a highly variable interface between the rock and overburden. The presence of cavities should be identified during the subsurface investigation. Consistent with RG 1.132, the spacing and depth of investigation locations should be reduced to detect the anticipated features.

A grouting program may be required to fill cavities and control seepage. The grouting program should include the potential to remove infilling from cavities using a water wash and fill the cavities as much as possible with grout. Replacing infill or open cavities with grout should increase and control variations in the rock mass modulus around and beneath the structures. Contact grouting is also required after construction of the shaft to avoid irregular external loading from voids – natural or due to overbreak during construction – on the exterior of the shaft. The rock surface may require modification through excavation or ground improvement to avoid significantly different stiffness along the shaft. Epikarst may form pinnacles or similar features that may result in variable stiffness along the shaft near the bedrock and overburden interface. The effect of potential cavities in the rock mass and variations at the bedrock and overburden interface on shaft deformation are evaluated on a site-by-site basis.

Table 3-4: Degradation Conditions and Criteria for Accessible Steel Structures

Degradation Condition	First-Tier Criteria	Second-Tier Criteria
Corrosion and/or corrosion stains	Absence of condition[1] [2]	Condition present, but determined acceptable after further review[3] [4] [5]
Bulges or depressions in liner plate	Absence of condition[1]	Condition present, but determined acceptable after further review[3]
Cracking/degradation of base or weld metal	Absence of condition[1]	Condition present, but determined acceptable after further review[3]
Leakage/Seepage (presence of water)	Absence of condition[1]	Condition present, but within original design limits of active leak-detection system and the leaking material and source do not present any adverse consequences[3]
Detached embedments or loose bolts	Absence of condition[2]	Condition present, but determined acceptable after further review [4]

[1] Section 5.1.2 of ACI 349.3R (Reference 8.18)

[2] Section 5.1.3 of ACI 349.3R (Reference 8.18)

[3] Section 5.2.2 of ACI 349.3R (Reference 8.18)

[4] Section 5.2.3 of ACI 349.3R (Reference 8.18)

[5] Section IWE-3500 of ASME XI (Reference 8.20) provides a threshold of 10% loss of nominal wall thickness.

3.4 Field Instrumentation Plan

Field instrumentation that beyond the current regulatory guidelines, is deployed to monitor the magnitude and distribution of pore pressure and amount of deformation during excavation, construction, loading and continuing through the BWRX-300 plant operation. The instrumentation provides recordings that can frequently be benchmarked against design estimates. Short-term and long-term settlement monitoring plans are developed that can detect both vertical and horizontal movements in and around the structures, as well as differential distortion across the foundation footprint and differential settlements between the CB, TB, RwB and RB foundations.

The specific locations of the sensors inside and outside of the RB shaft are dictated by the subsurface conditions and areas identified in the design where maximum stress, strain, and pore pressures are anticipated along the perimeter of the shaft. The definitive number of instruments is established during design stages of the monitoring system considering that the field instrumentation system shall be capable of:

- Measuring the rate of heave during excavation, especially at the end of excavation and at the bottom center and edges of the shaft.
- Measuring the rate of lateral displacement of excavation walls, throughout its depth, during and at end of excavation.
- Measuring the distribution of pore pressures around and below the RB shaft.

Mohr-Coulomb to other more sophisticated cases that incorporate strain-hardening/softening, strain dependent elastic and shear moduli, or rock failure criteria such as Hoek-Brown.

- Interface modeling described in Section 4.3.1; allows the introduction of the response and failure criteria between geometric zones; this feature is necessary to analyze faults, rock slip surfaces, or other discontinuities around the structure. The interface modeling has non-linear modeling capabilities.
- Interface modeling between soil/rock and structure described in Section 4.3.1; which is necessary to incorporate interaction between concrete and soil/rock via friction, accounting for the selected construction method and final configurations at the structure-soil/rock contacts. Non-linear behavior and separation are parts of the capability of this feature.
- Structure modeling, which may be limited to the main civil/structural components of the RB: main walls, floors, pools, and auxiliary structures.
- Soil/rock anchors and geogrids, which are used to simulate stabilization of the excavation and any associated potential failure surfaces.
- Fluid-soil interaction, which may be considered if the modeling the position of a static, horizontal groundwater table is not sufficient for the complexities in the design and construction of the BWRX-300 RB. Pore pressures are dependent on the permeability of the subsurface media, the hydrogeologic configuration, and the dewatering strategies for construction and operation.
- Staging analysis with time-dependent capabilities, which enables modeling the interaction of the structure and surrounding subgrade from excavation, through construction, loading and final operation. The model is capable of following stress/strain response as stress regimen changes from unloading during excavation to reloading after construction and during operation.

4.2 Subgrade Material Constitutive Models

Constitutive models define the relationship between the stresses and strains for different materials. Non-linear constitutive models are used for soils, rocks, and interfaces, or a combination of them.

The selection of the non-linear constitutive models for the BWRX-300 FIA is based on site-specific characteristics of the subsurface materials and the expected stress levels that result from dewatering, excavation, and loading. Regardless of the selected constitutive approach, the numerical model handles the potential for development of plastic zones or interfaces that can result from planes of weakness, presence of voids or cavities, or simple excess loading.

The parameters defining the soil and rock constitutive models are developed based on data obtained from the field and laboratory testing programs described in Section 3.1 and calibrated based on data collected from the field instrumentation program described in Section 3.4. This calibration includes modifying select input parameters for the soil and rock constitutive models or the interface models to better match the data collected from the field instrumentation program.

4.3 Non-Linear Foundation Interface Analysis Approach

The FIA addresses the following aspects:

- Interface modeling, described in Section 4.3.1, including both (a) contacts between structure and soil/rock, and (b) fault or joint planes or interfaces between bedding units in a geologic formation.
- Structural modeling of the main civil/structural components of the BWRX-300 and auxiliary facilities, described in Section 4.3.2, along with varying live and dead loads throughout the construction process.
- Fluid Soil Interaction, described in Section 4.3.3, to capture an adequate distribution of the space and time variation of pore pressures.
- BWRX-300 life stages: siting, excavation, construction, loading, and operation described in Section 4.3.4.

4.3.1 Interface Models

4.3.1.1 Interfaces Between the Structures and the Subgrade Media

The behavior of the contact at the base might not be critical for the RB because sliding and overturning are likely controlled by the deep embedment. However, the behavior of contact between the walls and soil, influences the soil pressures exerted on the structure along its embedded depth. The contact behavior depends on the selected construction methodology and changes through construction. For example, the contact condition of the BWRX-300 RB outer wall, when poured using a slurry wall or rock face as formwork, is different than the contact gained from a typical construction and backfill/grouting process. Figure 4-1 provides a schematic showing interfaces between structure and the surrounding media.

The interface is modeled, as is the case for the soil, with the use of an elastoplastic relationship based on an elastic deformation modulus and shear resistance. Figure 4-2 shows an example of interface rheologic modeling typically used for BWRX-300 FIA. A series of spring couplers are simulated at the connecting grid points at the interface. Each spring is represented by an elastoplastic model with Mohr-Coulomb criterion for shear failure.

When interface elements are used to represent the structure and soil/rock interaction, node pairs are created at the interface. From a node pair, one node belongs to the structure and the other node belongs to the soil/rock. The relative displacements (i.e. slipping/gap opening) can be simulated through elastic-perfectly plastic springs between these two nodes. Typically, two sets of springs are used for interface elements. One elastic-perfectly plastic spring to model the gap displacement and one elastic-perfectly plastic spring to model slip displacement. The simulation of gaps opening between the structure and soil/rock can be achieved through activating a tension cut-off for the spring that does not allow any tension at the interface.

The parameters of the slipping spring can be taken from the material set of the adjacent soil/rock elements or strength tests on natural and artificial discontinuities from the site investigation, laboratory testing program and characterization programs as described in Sections 3.1.2 and 3.1.3. The development of the interface parameters should be consistent with the limitations and modeling guidance of the software and interface model used for the nonlinear FIA. A strength

reduction factor can be used to adjust the spring stiffness based on the roughness of interaction and soil/rock residual strength when the sliding occurs. It is also possible to assign strength properties to interface elements based on direct measurements. If planar geosynthetic products are used during construction of the wall, shear properties are assigned to the interface elements representative of shear properties at geosynthetic/soil interfaces.

As is the case for soil and rock material constitutive models, the use of complex modeling capabilities for modeling interfaces introduces the challenge of identifying adequate input physical parameters. To address the uncertainties in these input parameters~~ in a conservative manner~~, ~~, the analysis may be conducted using bounding limits for the rheologic elastoplastic models assigned to the interface. One bounding scenario is a continuous connection case for which high stiffnesses (k) and soil equivalent failure criteria (ϕ, c) are assigned to the interface. S~~sensitivity ~~evaluations~~ analyses may be conducted ~~assuming lower friction and variations of the interface stiffness~~by adjusting ~~initial~~ spring stiffness and shear strength directly or through strength reduction factors. These types of analyses provide insight to understand the uncertainty introduced by interfaces in the stress distribution and deformation response of the structure.

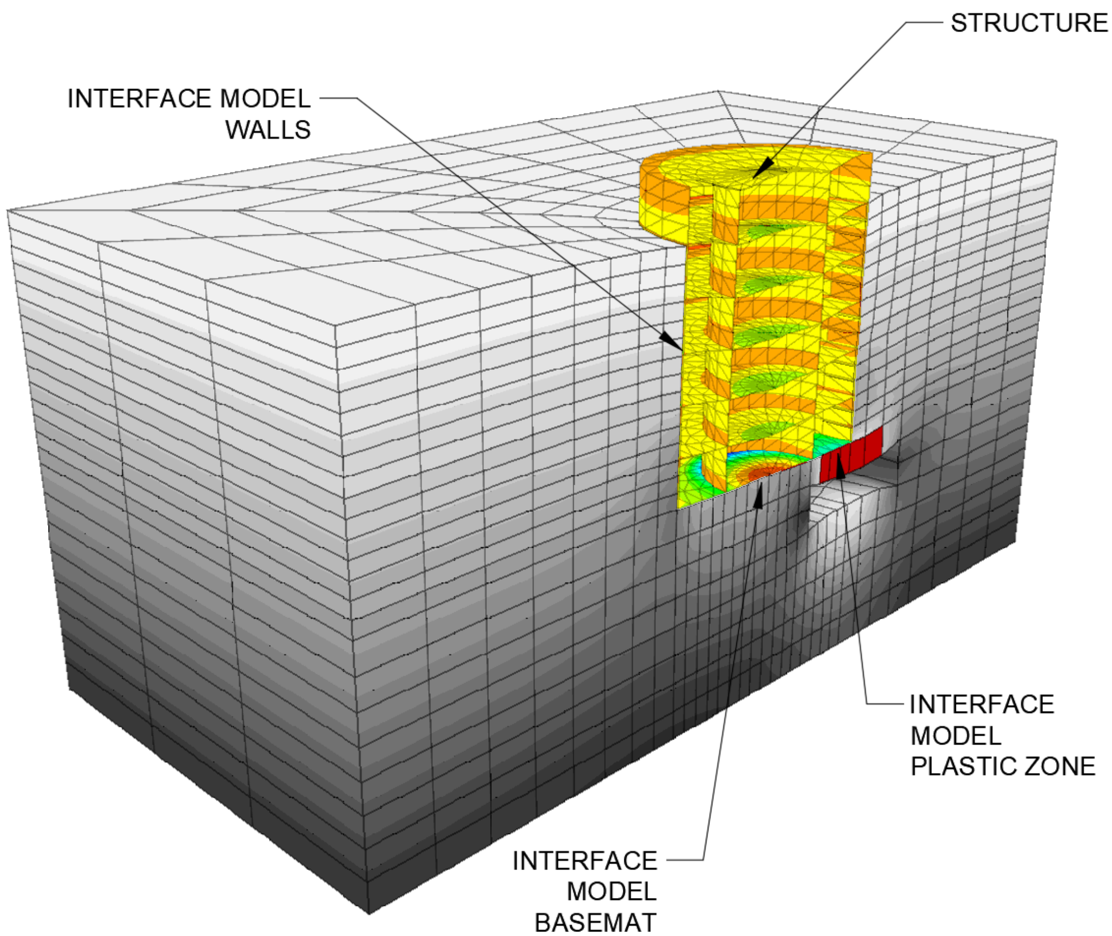

Figure 4-1: Location of Interfaces between Soil and Structure

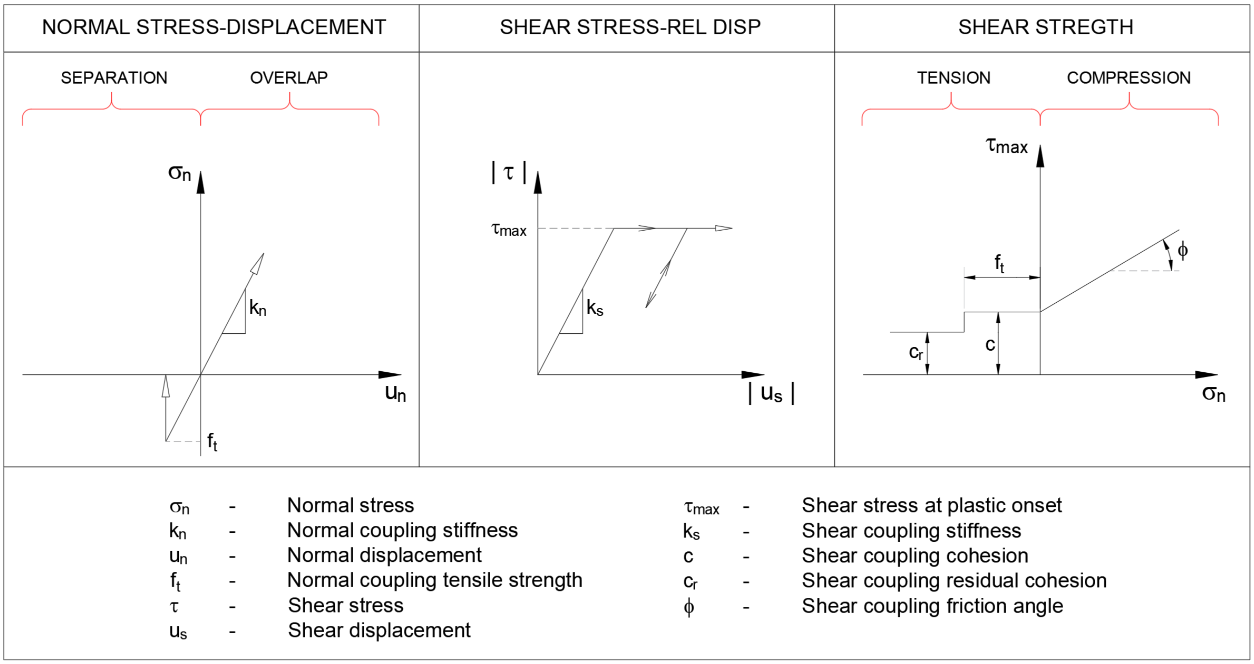

Figure 4-2: Interface Rheologic Modeling

4.3.1.2 Fault or Joint Planes or Interfaces Between Bedding Units in a Geologic Formation

The embedment depth of the BWRX-300 allows the possibility that soil rock interfaces, bedding interfaces, and other joints (Figure 4-3) may be in contact with the sides and base of the structure. These features may have planar or irregular configuration, and may be horizontal or with dipping, and even striking angles with respect to the position of the structure. The non-linearity and behavior of the joints are analyzed throughout the life stages of the reactor. These interfaces are modeled using similar interface modeling approaches as described in Section 4.3.1.1. The strength properties assigned to the interface elements along a rock discontinuity, i.e. bedding, are obtained from laboratory ~~or field~~ testing data described in Section 3.1.~~3~~2. ~~If~~When multiple strength tests are performed for rock discontinuities, ~~T~~the weakest strength parameters ~~representing the slipping may also be estimated based on the properties of the weakest interface material~~can be used for the interface elements or sensitivity analyses may be completed similar to Section 4.3.1.1. Strength reduction factors may be used to adjust the spring stiffness and shear strength based on the ~~roughness and residual~~ strength of the interface whe~~n~~re the sliding occurs.

operation. The model can simulate short-term as well as long-term dewatering or pumping as dictated by field conditions. The model simulates the changes in pore water pressures of the soil in response to unloading during the excavation stage and loading during construction and loading stages.

4.3.4 Analysis Staging Approach

Section 3.2 provides a description of the life stages of the BWRX-300, starting from the site investigation and ending with the plant operation. The BWRX-300 FIA are performed on numerical models that have the features to perform an integrated analysis of the stress, and deformation fields for each of the identified life stages:

4.3.4.1 Site Characterization

The FIA begins with the site itself, in its native condition, prior to any excavation or construction activities. During this stage, the initial stress conditions are aligned with the initial baseline displacement field. Initial stress conditions include, if applicable, the influence of groundwater aquifers and measured horizontal stresses.

4.3.4.2 Excavation

During the BWRX-300 RB shaft excavation, shown on Figure 4-4, soils and rock around and below the shaft may experience tensile stresses. The selected constitutive models allow for expansion response of soils resulting in heave or added pressures on excavation support structures. The changes in site conditions made prior or during the excavation are introduced in the FIA model following the sequence of the excavation plan. Non-linear interfaces are modeled between stabilization walls and soil.

As shown on Figure 4-5, the excavation simulation resembles the scheme planned for the specific site, by staging the removal of soil layers as excavation progresses and excavation support and site improvements are made. The stability of the excavation is verified in analytical space and later compared against field observations. The process allows for the design and monitoring of a safe excavation.

At the end excavation, the stress and displacement fields of the surrounding media, as well as the distribution of pore pressure, will have evolved. The "after excavation" condition is used as the initial condition for the analysis of the construction stage.

- groundwater hydrostatic pressure; and
- overburden loads and the interaction with the surrounding RwB, CB and TB foundations and structures.

Furthermore, the interaction with the surrounding subgrade determines the boundary conditions at the RB below-grade shaft exterior wall and basemat interfaces thus affecting the structural response and stress distribution from other static and dynamic loads such as operating and accidental thermal and pressure loads.

In order to adequately account for the SSI effects, the one-step approach, as defined in Section 3.1.2 of ASCE/SEI 4-16 (Reference 8.7), is implemented for the design of the BWRX-300 RB structure using a linear elastic SASSI (a system for analyses of soil-structure interaction) analysis approach described in Section 5.3. Static and dynamic structural stress demands are obtained directly from the results of SSI analyses of combined models that include FE representations of the RB structure and the surrounding soil. The surrounding subgrade is represented by layered half-space continuum with equivalent linear elastic stiffness properties and complex damping.

Stress demands on the RB structural members due to static earth pressure, structural self-weight, equipment weight and life loads are calculated by applying 1-g gravity loads on the combined model of the RB structure and the subgrade continuum. The structural demands due to overburden pressures from the nearby foundations are also calculated by the 1-g static analysis. Additional static analyses are performed to calculate the structural demands due to hydrostatic wall pressures from the pool water, normal operating and accidental pressure loads. Separate analyses provide the structural demands due to normal operating and accidental pressure and thermal loads. Structural demands due to seismic inertia loads and dynamic soil pressure loads are obtained from seismic SSI analyses that are described in Section 5.3.

The methodology used for development of RB FE model is based on the methodology described in Section 5.1.1 and the SSI modeling assumptions presented in Section 5.1.2. Equivalent linear properties are used as input for the static and seismic SSI analyses developed as described in Sections 5.2.1 and 5.2.4, respectively. Section 5.1.3 presents the unique BWRX-300 approach used to demonstrate that the linear-elastic SSI analyses provide soil and rock pressure load demands with sufficient design load margins to address the modeling uncertainties.

5.1.1 FE Model of RB Structure

The structural FE model consisting of beam, shell, solid, and spring elements adequately represents the RB structural configuration for all main structural members. The FE model includes gross discontinuities such as large openings and member eccentricity. Thick shell elements are used to model the reinforced concrete shear walls, slabs and basemat. 3-D beam elements are used to model the reinforced concrete or steel columns, beams, and trusses. The shell and beam elements are established at the centerline of the wall, slab, beam, column, and truss elements. Rigid beam and shell elements or rigid links are used to model member eccentricities and offsets.

Linear elastic contact springs connect the RB structural and subgrade FE models. Stiffness properties are assigned to the contact springs to adequately represent the interaction mechanism between the structure, the water proofing material and the soil as described in Section 5.1.2.

Results obtained from these contact spring elements serve for calculation of soil pressures on the below grade RB shaft exterior wall. The results obtained from the contact spring elements serve to:

- validate the earth pressure loads considered by the design as described in Section 5.1.3, and
- determine whether separation between RB shaft wall and soils occurs in the static and dynamic loadings as discussed in Section 5.3.9.

The mesh of the FE models is sufficiently refined to produce stress demand calculations that are not significantly affected by a further refinement of the FE size or the shape. Finer meshes are used around penetrations and openings that are larger than half of the wall or slab thickness. Meshes of major walls and slabs consists of at least four shell elements along the short direction and at least six shell elements along the long direction.

The FE models used for seismic SSI analyses have a sufficiently refined mesh to be capable of transmitting the entire frequency range of interest for the seismic design of the RB SSCs. In accordance with the requirements of ASCE\SEI 4-16 (Reference 8.7), Section 5.3.4, the FE mesh shall be smaller than or equal to one-fifth of the smallest wavelength transmitted through the soil model, i.e. the maximum mesh size:

$$d_{max} \leq \frac{V_S}{5\, f_{cutoff}} \qquad (5\text{-}1)$$

where: V_S is the shear wave velocity of the transmitting soil material; and

f_{cutoff} is the cutoff frequency of analysis determined as described in Section 5.3.2

Larger element sizes may be used when justified as described in Section 5.3.4 of ASCE\SEI 4-16. Stiffness properties are assigned to structural members in the RB FE model in terms of Young's modulus and Poisson ratio that are determined in accordance with the governing design codes:

- American Concrete Institute ACI-349-13 (Reference 8.24) for the reinforced concrete members; and
- AISC N690-18 (Reference 8.25) for the steel and steel-plate composite (SC) members.

5.1.2 Soil-Structure Interaction Modeling Assumptions

Several simplified assumptions are introduced in the SSI design analyses of RB FE model to enable an efficient calculation of stress demands on the RB structure due to pressure loads from soil and rock surrounding and supporting the RB shaft. The following are the main ~~SSI modeling~~ assumptions for subgrade modeling used for the design SSI analyses performed following the SASSI methodology:

1) The properties of the subgrade materials are assumed to be isotropic and linear elastic;
2) The non-linearities at soil-structure interfaces are neglected;

3) The rock mass is assumed continuous and the presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones is neglected;

4) The static lateral pressures on the RB shaft due to the weight of self-supporting rock (i.e., excavated rock that does not require lateral support) can be neglected. ~~The rock is assumed self-supporting, i.e. no lateral support is required of the excavated rock.~~

As described in Section 5.2.1, an approach is used for the development of linearized properties of soil and rock materials for the 1-g static SSI analysis to provide upper bound estimates of the demands on the RB structural members. Upper bound structural deformations and stress demands and lateral soil pressures on the RB below-grade exterior walls are estimated by using upper bound values for the soil unit weight and soil and rock Poisson's ratio paired with lower bound values of soil and rock elastic moduli.

The following stiffness properties are assigned to the contact springs at the SSI interfaces in the RB FE model for 1-g design analysis to provide upper bound lateral soil pressures on the RB below-grade exterior walls:

- The contact springs in the direction normal to the RB exterior walls are assigned properties representing upper bound stiffness conditions at the SSI interfaces; and
- The friction at the RB exterior walls is neglected by assigning very low stiffness properties to the contact springs in vertical and tangential direction.

The soil and rock strata in the SSI models used for calculating demands for design of RB structure are modeled based on the principles of continuum mechanics using isotropic linear elastic properties. Possible fracture zones, joints, bedding planes, discontinuities and cavities in the rock are not explicitly included in the design SSI analyses models. The stiffness properties assigned to the rock materials are developed, as described in Section 5.2.1.2, using empirical engineering and geomechanical rock mass classifications that quantitatively characterize the geologic and engineering parameters of rock masses.

The approaches described in Section 5.2.1.2 to calculate the equivalent linear properties of rock are applicable to structures that are relatively large compared to the block size of the rock mass and assumes the closely spaced discontinuities have similar characteristics where isotropic behavior of the rock mass is valid. When the discontinuity spacing is large compared to the dimensions of the excavation, the potential for unstable blocks or wedges and swelling or squeezing rock units need to be evaluated. The size of potentially unstable rock blocks and wedges should be estimated using an appropriate method (e.g., Reference 8.69). The evaluation of the potential loads from rock blocks and wedges may be completed using:

- ~~-~~the nonlinear FIA that includes rock/rock discontinuities represented by interface models described in Section 4.3.1.2; or~~simple~~
- static or pseudostatic force equilibrium analysis.

A simple example of a model for force equilibrium analysis of rock stability is provided in Section 5.1.4.3.

Strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness ~~will frequently~~may be self-supporting even if some reinforcement is

required to ensure a safe excavation. Typically, rock masses will yield slightly during construction – even with well-placed reinforcement – and arching will reduce the lateral loads except in highly fractured, weak, swelling, or squeezing rocks. ~~Joints and other weak planes may create isolated blocks that are unstable; however, these blocks are not typically large relative to the area of the structure and would be unlikely to produce significant loads on the exterior of the structure compared to other loads (e.g., hydrostatic). These blocks would also not be able to create a cascading failure once the structure is in place.~~

Because it is much more economical to reinforce the rock mass than to support it, rock reinforcement is used to create a self-supporting rock mass when the natural rock mass is not self-supporting. Reinforcement like tensioned and untensioned anchors may be installed inside the rock mass to help the rock mass support itself by eliminating progressive failure along planes of low strength as described in USACE 1110-1-2907 (Reference 8.26). Frequently, the reinforcement addresses specific rock wedges (keying) or is designed to form a beam or arch within the rock to create a stable, self-supporting excavation. Surface treatments such as shotcrete, strapping, and mesh may also be used for stabilization, protection of exposed rock, and control of loosened rock.

The design of the BWRX-300 considers this rock reinforcement as initial ground support that is separate from the permanent ground support system because the rock reinforcements and any surface protection may be inaccessible after construction. Therefore, the design addresses the rock loads remaining after the initial ground support degrades by including the potential weight of the solid rock in the design 1-g SSI analysis based on the results of non-linear FEA as described in Section 5.1.3.

Additional design analysis may be performed where earth pressure loads are applied to the below grade exterior walls of the refined RB structural model to account for:

- the effects on the RB design of anisotropic or heterogenous rock responses that cannot be directly modeled by the isotropic elastic models used for the one-step design SSI analysis; or
- potential pressures from unstable blocks of rock mass.

The magnitude and distribution of these additional earth pressure loads are determined from the results of the nonlinear FIA or force equilibrium analyses of the unstable rock mass. The structural design demands obtained from this additional earth pressure analysis are combined with the results of the one-step SSI analysis to ensure the RB structural design adequately addresses the effects of anisotropic and heterogenous rock behavior and accounts for potential unstable rock mass loads.

The SSI analysis of RB FE model are performed for a set of subgrade profiles to account for the variability and uncertainties in the subgrade material properties in accordance with the regulatory guidance of SRP 3.7.2 Subsection II.4 and ASCE/SEI 4-16 (Reference 8.7), Section 5.1.4. To address the effects of primary non-linearity, soil dynamic properties are used that are compatible to the free-field strains generated by a typical design level earthquake. These strain-compatible properties are developed as described in Section 5.2.4.

The effects of secondary non-linearity induced in the soil and rock by the structural vibration are neglected because in general, the structural vibration induces plastic deformations of the soil and dissipation of energy in the SSI system that reduces the structural response as shown in

protection will be inaccessible for monitoring and repair after the construction. Therefore, unimproved soil and rock conditions are considered due to the uncertainty in:

- the long-term durability of grout, as noted in Paragraph 2-5 of USACE EM 1110-2-3506 (Reference 8.29);
- potential degradation of rock reinforcement, as noted in USACE EM 1110-1-2907 (Reference 8.30); and
- degradation of other soil support system.

This additional rock load on the RB shaft wall may be uniform with contact grouting to avoid stress concentration or point load associated with the block or wedge that is reinforced to stabilize the rock excavation. The evaluation of these rock pressure loads assumes that the excavation has reached stability with initial rock support and that the liner will accept 100 percent of the initial rock support as it relaxes over the lifetime of the structure. These loads should be conservative because rock loads in stressed rock masses are typically not following (e.g., they are not independent of displacement and typically reduce with displacement due to arching). The notable exception would be due to the presence of hydrostatic loads and swelling or squeezing rock displacements that may continue to apply a large load with continued displacement.

The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. This potential load transfer is dependent on the geometry of the discontinuities, surface structure and embedded structure. When the additional load from the surface structure may be transferred to a potentially unstable rock block or wedge, this additional load should be included in the determination of reinforcement and the potential rock load on the exterior of the shaft or the rock block or wedge may be over-excavated and backfilled to reduce the load. Consideration of the geometry of the load transfer may allow the surface structures to be re-arranged to reduce or eliminate this load transfer to a potentially unstable rock block or wedge.

If cavities are present at the deployment site, sensitivity analysis are also performed by varying locations and sizes of cavities to address the effects of potential cavities on the rock pressure demands on the RB structure during operation.

The pressure load validation FIA uses the constitutive models described in Section 4.2 to represent the non-linear response of soil and rock subgrade materials, and the models described in Section 4.3.1 to represent the response at interfaces including the interfaces of RB structure with the surrounding subgrade. Because the intent of the FIA is to calculate best estimates of the soil and rock pressure loads, constitutive and interface models are developed using best estimate soil and rock properties obtained from the results of site investigation and laboratory testing programs described in Section 3.1. The stiffness of the RB structure in the FIA models is calculated per the governing design codes. Conservative design values obtained from the literature can also be used for certain input parameters.

A best estimate soil and rock pressure profile on the RB shaft is developed as an envelope of all maximum lateral pressure values calculated by the non-linear FIA of all analyzed post-construction stages and scenarios. This lateral pressure profile is compared to the lateral pressure profile developed from the results of the linear elastic 1-g design analysis to confirm the

equivalent linear elastic model provides adequately conservative loads for the structural design. Soil and rock design pressure margins are calculated based upon the minimum values and the distribution of the ratio between the design soil and rock pressures obtained from the 1-g linear elastic analysis and the best estimate pressures obtained from the non-linear FIA. If the values of the calculated soil and rock design load margins are below the values deemed adequate to address the uncertainties and variations of subgrade properties, the rock mass weight or the equivalent linear soil and rock stiffness properties used for the 1-g design analysis are adjusted. Adequate values of the soil and rock design load margins are established based on the uncertainties and variability of soil and rock properties used as input for the non-linear ~~FEA~~ FIA and the significance of the non-linear and anisotropic response of subgrade materials on the soil and rock pressure demands.

If the results of non-linear static FIA indicate that the non-linear and anisotropic effects have a significant effect on the rock soil pressures and the site is characterized by a high seismicity, sensitivity SSI analyses are performed on non-linear models, as described in Section 5.3.11, to assess the effects of non-linear soil and rock response on the dynamic lateral pressure demands.

5.1.4 Probabilistic Earth Pressure Analyses

Probabilistic analyses may be performed to demonstrate that the magnitude of earth pressures used for the design are adequate to address uncertainties in the pressure load calculations. The external wall of the RB that is contact with soil is subdivided into discrete regions. The general approach consists on computing the probability density function of the subgrade pressure at each discrete region to calculate the probability distributions of soil and rock pressure loads on the RB below-grade exterior walls.

The probabilistic earth pressure load analysis addresses two types of uncertainties in the calculations of earth pressure loads:

- Parameter uncertainties related to natural randomness and uncertainties in measurements of mechanical properties of in-situ subgrade materials; and
- Model uncertainties related to the models used for earth pressure calculations.

Parameter uncertainty includes random variability of measured parameters including spatial variability and systematic measurement errors as well as uncertainties related to the methods used for the development of site subgrade parameters from empirical relationships. The random variability is manifested as the scatter of the data around a mean trend and is composed of the spatial variation of the subgrade properties and random measurement errors. Because the random measurement errors are often not distinguishable from spatial variation of the subgrade properties, they are usually considered jointly. Systematic error is divided into:

- Statistical error in the mean that can be reduced with increasing the sample size and number of measurements and tests being performed
- Bias in sampling and measurement procedures that is corrected by means of correction techniques/algorithms

- Bias introduced by the method used for development of subgrade parameters that is addressed by considering different approaches and empirical equations to calculate discrete probability distributions that are then combined as described in Subsection 5.1.4.4.

The model uncertainty that represents the uncertainty related to the model's ability to accurately predict the soil and rock pressures is manifested as a bias error in the earth pressure calculations. In general, the model uncreate is reduced by using more sophisticated models and an increasing number of model parameters. On the other hand, the increasing number of parameters used in the sophisticated models increases the parameter uncertainty and may reduce the overall confidence in the calculated soil pressure results. The model uncertainty is approached by means of considering different models that utilize fewer input parameters resulting in discrete probability distributions that are combined as described in Subsection 5.1.4.4.

5.1.4.1 First Order Second Moment Method

The First Order Second Moment (FOSM) method may be used for simple calculations of the probability density function of the ground pressure. Following the approach described in (Reference 8.31), earth pressures (P) at each discretized region are represented by the following function:

$$P = g(x_1, x_2 \dots x_n) + e \qquad (5\text{-}2)$$

where: g represents a geotechnical multivariable function of the earth pressure at a discretized element

$x_1, x_2 \dots x_n$ are the site parameters whose variation has an important effect on the earth pressures

e represents the biased modelling and measurement systematic errors.

The probability calculations may consider other parameters than the random parameters $x_1, x_2 \dots x_n$. These parameters where variations have relatively insignificant effects on the earth pressures, may be considered deterministically using values that ensure a reasonably conservative bias in the results of the probabilistic analyses.

The mean value of the earth pressure ($\bar{P}$) is expressed as function of the mean values of the site parameters ($\bar{x}_1, \bar{x}_2 \dots \bar{x}_n$):

$$\bar{P} = g(\bar{x}_1, \bar{x}_2 \dots \bar{x}_n) \qquad (5\text{-}3)$$

For a sample of $1, 2 \dots m$ measurements, the mean values of each parameter $\bar{x}_i$ in Equation (5-3) are calculated as follows:

$$\bar{x}_i = \frac{1}{m} \sum_{k=1}^{m} (x_{ik}) \qquad (5\text{-}4)$$

where: x_{ik} is the kth measured data point of the parameter x_i.

- a FE model or a finite difference model.

Table 5-1 summarizes the different site parameters and types of models that are commonly used in the probabilistic analyses of earth pressures, in particular for the FSOM calculations to obtain the values of parameter derivatives dg/dx_i.

Table 5-1: Models for Probabilistic Earth Pressure Analyses

Subgrade Type	Site Parameter (x_i)	Model
soil	unit weight	Analytical equations
	cohesion	
	friction angle	
rock	rock mass properties	Force equilibrium, FE or a finite difference model
	unit weight	
	cohesion	
	friction angle	
	weak zone orientation	
	weak zone area	

Simple models that do not require explicit calculations of the state of strain and stress in the ground materials, are used for the probabilistic analyses of earth pressures on the RB shaft in contact with subgrade materials which mechanical properties are assumed to be continuous. For example, the following three models can be used to calculate lateral earth pressure coefficients representing three possible states:

a. at-rest condition representing essentially no movement of the structure relative to the surrounding subgrade;

b. active condition when the structure moves away from the surrounding subgrade; and

c. passive condition when the structure moves towards the surrounding subgrade.

These simple models provide probabilistic earth pressure distributions from the probabilistic distributions of the basic subgrade material strength parameters, the internal friction angle (φ), the cohesion (c) and the friction angle (φ_w) between the subgrade and RB cavity wall.

Force equilibrium models are used for probabilistic analysis of rock masses with discontinuities that may control the stability of individual blocks or the rock mass when the orientation is disadvantageous. Depending on the geometry of the discontinuities relative to the free face of the excavation, one or more blocks may slide along the discontinuities.

As shown on Figure 5-1, the sliding of the rock block driven by the surcharge load and its own weight is resisted by:

8.47 U.S. Nuclear Regulatory Commission, "Seismic Input and Soil Structure Interaction Final Report," NUREG/CR-0693, February 1979.

8.48 Brookhaven National Laboratory, "Assessment of Seismic Analysis Methodologies for Deeply Embedded Nuclear Power Plant Structures," U.S. Nuclear Regulatory Agency, Washington, D.C., NUREG/CR-6896/ BNL-NUREG-75410-2006, ADAMS Accession No. ML060820521, February 2006.

8.49 Wolf, J. P., "Dynamic Soil-Structure Interaction," Prentice-Hall, Inc., Englewood Cliffs, NJ 07632, 1985.

8.50 NEI White Paper, "Consistent Site-Response/Soil-Structure Interaction Analysis and Evaluation," Nuclear Energy Institute, 2009.

8.51 EPRI Report 3002009429: "Advanced Nuclear Technology: High-Frequency Seismic Loading Evaluation for Standard Nuclear Power Plants," Electric Power Research Institute, Palo Alto, CA, 2017.

8.52 Tseng, W.S., "Equipment Response Spectra Including Equipment-Structure Interaction Effects", ASME PVP Conference, Honolulu, 1989.

8.53 IBC, "International Building Code," International Code Council, 2018.

8.54 ASCE 7-16, "Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers," 2016.

8.55 ACI 318-14, "Building Code Requirements for Structural Concrete, American Concrete Institute," 2014.

8.56 ICC 500, "ICC/NSSA Standard for the Design and Construction of Storm Shelters," International Code Council, 2014.

8.57 ASCE/SEI 41-17, "Seismic Evaluation and Retrofit of Existing Buildings," 2017.

8.58 ANSI/AISC 360-16, "Specification for Structural Steel Buildings, American Institute of Steel Construction," 2016.

8.59 Todorovski, L., W. Silva, D.M. Ghiocel, and K. Lanham, Generic Input for Standard Seismic Design of Nuclear Power Plants, SMiRT-22, 2013.

8.60 EPRI-102293: "Guidelines for Determining Design Basis Ground Motions," Electric Power Research Institute (EPRI), Palo Alto, CA, 1993.

8.61 Iowa DOT 200E-1, "Geotechnical Design: Engineering Properties of Soil and Rock," Iowa Department of Transportation (DOT) Office of Design, Chapter 200, May 2015.

8.62 Lambe, T.W. and R.V. Whitman, "Soil Mechanics," Massachusetts Institute of Technology, John Wiley & Sons, 1969.

8.63 FHWA NHI-16-072: "Geotechnical Site Characterization," U.S. Department of Transportation, 2017.

8.64 RG 1.132, "Site Characterization Investigations for Nuclear Power Plants," Revision 2, October 2003.

8.65 RG 1.138, "Laboratory Investigations of Soil and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3, December 2014.

8.66 American Society for Testing and Materials ASTM D5607, "Standard Test Method for Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force," 2016.

8.67 USACE, "Method of Test for Direct Shear Strength of Rock Core Specimens," RTH 203-80, Waterways Experiment Station, Vicksburg, MS, 1993.

8.68 USACE, "Standard Method of Test for Multistage Triaxial Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements," RTH 204-80, Waterways Experiment Station, Vicksburg, MS, 1993.

~~8.64~~8.69 Goodman, R.E., and Shi, G., Block Theory and its Application to Rock Engineering, Prentice-Hall, Inc, 1985.

Appendix B
Replaced Pages from NEDO-33914, Revision 1

REVISION SUMMARY

Revision Number	Description of Change
0	Initial Revision
1	Revised to incorporate the following responses to NRC Requests for Additional Information (eRAIs): • NRC eRAI 9849 Question 01.05-01 revised Sections 1.3, 2.4, 6.1, 6.1.2, and 6.4 to reflect the relocation of the offgas system charcoal adsorbers to the Radwaste Building. • NRC eRAI 9859 Question 02.05.04-01 revised the following sections: o Section 3.1.2 to include direct shear and triaxial strength tests for natural and artificial discontinuities. o Section 3.1.3 to identify direct shear and triaxial compressive tests as laboratory tests on recovered samples of discontinuities. o Section 4.3.1.1 to state that the interface parameters are from adjacent soil/rock elements or strength test on natural and artificial discontinuities developed to be consistent with the selected nonlinear FIA software and interface model. o Section 4.3.1.2 to indicate that the weakest strength parameters from multiple tests on rock discontinuities may be used for interface elements. o Section 8.0 to add references for ASTM D5607, RTH 203-80, RTH 204-80, RG 1.132 and RG 1.138. • NRC eRAI 9859 Question 02.05.04-02 revised the following sections: o Section 3.1.1 to identify the need for geologic mapping of outcrops to improve the field investigation program and the rock characterization when bedrock is encountered at depths for engineering purposes. o Table 3-1 to add "characterize rock mass and discontinuities". o Section 3.1.3 to identify the investigation locations and methods, including geologic mapping, intended to characterize the rock and rock mass parameters. • NRC eRAI 9859 Question 02.05.04-03 revised the following sections: o Section 3.4 to indicate that the specific locations of sensor may be inside and outside of the RB shaft.

Revision Number	Description of Change
	o Section 4.2 to include a description of the calibration process using parameters for the selected soil and rock constitutive models. o Section 5.1 to clarify that the one-step approach is implemented using a linear elastic SASSI analysis approach. • NRC eRAI 9859 Question 02.05.04-04 revised Assumption (1) in Section 5.1.2 and also revised Section 5.1.2 to describe the approach used to account for the effects of anisotropic or heterogenous rock response including potential pressures from unstable blocks of rock mass. • NRC eRAI 9859 Question 02.05.04-05 revised the following sections: o Section 5.1.2 to remove the text stating that isolated unstable blocks do not produce significant loads. o Section 5.1.3 to include other mitigation methods like overexcavation and backfilling when the potential load from a block or wedge is large. o Section 8.0 to include Goodman and Shi (1985) as a reference. • NRC eRAI 9859 Question 02.05.04-06 revised Assumption (4) in Section 5.1.2. • NRC eRAI 9859 Question 02.05.04-07 revised Section 3.1.3 to discuss characterization of the groundwater conditions. • NRC eRAI 9859 Question 02.05.04-08 revised the following sections: o Section 3.1.1 to define d_{max}. o Table 3-1 to correct a typographical error. o Section 3.1.3 to include a discussion on reviewing the current knowledge of the state of stress in the bedrock. o Section 4.3.4.1 to include the influence of groundwater and measured horizontal stresses in the initial stress conditions for the nonlinear FIA. • NRC eRAI 9859 Question 02.05.04-09 revised the following sections: o Section 5.1.4 to explain that the probabilistic earth pressure evaluations also consider uncertainties in the methods or empirical relationships used for development of site related parameters. o Table 5-1 to indicate that the models for probabilistic analyses consider the natural and measurement uncertainties in rock mass properties.

Revision Number	Description of Change
	Added reference citations to the following sections for RG 1.132 and 1.138: 1.1, 2.1, 2.5, 3.0, 3.1, 3.1.1, 3.1.2, 3.1.3, and 3.5. Corrected typographical errors in Sections 1.3, 3.0, 5.1.2, and 5.1.3. Corrected lettered bullets so they start with A in the following sections: 3.2.1, 3.4, 4.1, 4.3, and 7.3.

1.0 INTRODUCTION

1.1 Purpose

The purpose of this report is to present design, analysis, and monitoring guidelines and requirements for construction of a BWRX-300 Small Modular Reactor (SMR) using innovative and comprehensive approaches that ensures safe operation throughout the life of the plant. The BWRX-300 innovative methodologies and approaches meet 10 CFR 50, Appendix A, General Design Criteria (GDC). Further, the innovative approaches presented herein meet the intent of NUREG-0800 "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition" for large light water reactors and other NRC guidance while addressing specific areas of concern identified in NUREG/CR-7193, "Evaluations of NRC Seismic-Structural Regulations and Regulatory Guidance, and Simulation-Evaluation Tools for Applicability to Small Modular Reactors (SMRs)," (Reference 8.1) for the design of deeply embedded SMRs.

As described in Section 1.3, a cost-effective design concept is implemented for the BWRX-300 where the majority of important safety- related systems and components are located in the below grade Reactor Building (RB) vertical right cylinder shaft. The cost for construction of the BWRX-300 RB is optimized by minimizing the amount of excavation and reducing the amount of backfill, as discussed in Section 1.4. The construction method is provided as relevant background information to identify the effects of deep excavation and construction sequences on site characterization, soil properties, and methodology used to analyze and design the BWRX-300 RB structure, including construction monitoring and inspections.

The following criteria, methodologies, recommendations, and approaches specific to the innovative BWRX-300 design are addressed in the report and may be referenced during future licensing activities either by GEH in support of a 10 CFR 52 Design Certification Application (DCA) or by a license applicant requesting a Construction Permit (CP) and Operating License (OL) under 10 CFR 50 or a Combined Operating License (COL) under 10 CFR 52:

A. Requirements and recommendations are provided in Section 3.1 for site investigation and subsurface materials laboratory testing programs that address the specific BWRX-300 configuration with the RB vertical shaft deeply embedded in in-situ soil and/or rock materials. The provided recommendations are beyond current regulatory guidance for large light water reactors and define additional requirements for characterizing in-situ materials surrounding the deeply embedded SMRs. These additional requirements address current limitations in Regulatory Guide (RG) 1.132 "Site Investigations for Foundations of Nuclear Power Plants," Revision 2 (Reference 8.64), and RG 1.138 "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 3 (Reference 8.65), that could adversely affect the results when applied to deeply embedded SMRs, as identified in NUREG/CR-7193, Sections 1.5.3 and 1.5.5.

B. Methodologies and approaches for non-linear Foundation Interface Analyses (FIA) are recommended in Section 4.0 that are supported by the results from the data collected from the inspection and monitoring programs described in Sections 3.2 and 3.3. This innovative approach ensures, with a high level of confidence, that the stability of the deeply embedded BWRX-300 RB structure will be maintained throughout the life of the plant and addresses

deeply embedded SC-I RB structure. CB, TB and RwB are separated from the RB by seismic gaps. The CB houses the control room, electrical, control and instrumentation equipment. RwB houses rooms and equipment for handling, processing, and packaging liquid and solid radioactive wastes. TB encloses the turbine generator, main condenser, condensate and feedwater systems, condensate purification system, off-gas system (OGS) cooler and refrigerant dryer, turbine-generator support systems and bridge crane.

The RwB, which houses the systems for management of radioactive gas, liquid and solid radiological waste is categorized as RW-IIa in accordance with Regulatory Guide (RG) 1.143 "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants," Revision 2.

The CB, TB and RwB structures are evaluated as described in Section 7.0 to prevent structural failure or interaction that could:

- degrade the functioning of the RB SC-I SSCs to an unacceptable level of safety;
- result in incapacitating injury to occupants of the CB control room; and
- compromise the safety functions of those SSCs that are required to remain functional following a seismic event.

Figure 1-3 shows a conceptual RB section view for the BWRX-300. This drawing is provided for information only and does not reflect the final BWRX-300 design.

1.4 Reactor Building Below-Grade Shaft Construction

The construction process determines the final configuration of the interface between the RB below-grade shaft and surrounding soil and/or rock and defines the scope of the analysis and design of the RB. The construction method also influences the scope and extent of subsurface site investigations in addition to the inspections and monitoring requirements during construction and throughout the life of the plant through decommissioning.

For most subsurface conditions, traditional methods for excavation and construction of the BWRX-300 RB translate into prolonged schedules and high costs. To optimize the cost of the BWRX-300, innovative approaches are employed for the construction of the RB below grade shaft that are aimed to:

- minimize the amount of excavation;
- reduce the amount of engineered backfill; and
- reduce construction schedule.

The construction approach for the BWRX-300 will require adaptation to the site-specific conditions. Most physical subsurface settings will consist of configurations that include soil overburden and rock beneath. For purposes of illustrating the construction approaches, a generic transition profile is considered having the upper two thirds with soft soil strata followed by a lower third with hard rock strata. This transition profile is selected to include the required transition in construction techniques from soil to rock subgrade conditions.

2.0 REGULATORY BASIS

This section describes compliance with regulatory requirements and the bases for any exemptions to regulatory requirements or approaches to regulatory guidance that may be referenced during future licensing activities either by GEH in support of a 10 CFR 52 Design Certification Application (DCA) or by a license applicant requesting a Construction Permit (CP) and Operating License (OL) under 10 CFR 50 or a Combined Operating License (COL) under 10 CFR 52.

10 CFR 50, Appendix B establishes quality assurance requirements for the design, manufacture, construction, and operation of nuclear power plant SSCs that prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public. The pertinent requirements of 10 CFR 50 Appendix B apply to all activities affecting the safety-related functions of those SSCs.

10 CFR 50, Appendix B establishes in Clauses X and XI quality assurance requirements for the design, construction, and operation of nuclear power plant SSCs that prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public.

The following is the regulatory basis specific to the innovative approaches implemented for the analysis, design, construction, and maintenance of the BWRX-300 important to safety structures. The implemented innovative approaches meet the intent of the current regulatory guidance for large light water reactors and address the specifics related to the seismic and structural design of deeply embedded SMRs identified in NUREG/CR-7193 (Reference 8.1).

2.1 Regulatory Basis for Defining Site Subsurface Conditions

10 CFR 100 requires the consideration of site physical characteristics, including seismology and geology. 10 CFR 100.20(c)(1) and 10 CFR 100.23 establish requirements for conducting site investigations for nuclear power plant license applications.

IAEA Safety Guide NS-G-6 provides guidance on the methods and procedures for analyses to support the assessment of the geotechnical aspects for the design of nuclear power plants.

SRP 2.5.4 provides regulatory guidance for the investigation and reporting site-specific geologic features and characteristics of ground materials, including static and dynamic engineering properties and groundwater conditions.

RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Revision 2 (Reference 8.64), describes methods acceptable to the NRC staff for conducting field investigations to acquire the geological and engineering characteristics of the site and provides recommendations for developing site-specific guidance for conducting subsurface investigations. Details regarding the technical bases described in RG 1.132 are provided in NUREG/CR-5738 "Field Investigations for Foundations of Nuclear Power Facilities" (Reference 8.2).

RG 1.138 (Reference 8.65) describes laboratory investigations and testing practices for determining soil and rock properties and characteristics needed for engineering analysis and design of foundations and earthworks for nuclear power plants. NUREG/CR-5739 "Laboratory Investigations of Soils and Rocks For Engineering Analysis and Design of Nuclear Power Facilities" (Reference 8.3), provides the technical basis for the techniques for laboratory testing of

The basis for the seismic design of BWRX-300 RB SSCs is developed based on the results of SSI analyses performed following the regulatory guidance of SRP 3.7.2 and DC/COL ISG-01 (Reference 8.8), and in accordance with the ASCE/SEI 4-16, Section 5 provisions (Reference 8.7).

The SSI analyses are performed on finite element (FE) models of RB that are developed in accordance with the regulatory guidance of SRPs 3.7.1 and 3.7.2, and RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," Revision 1, and the provisions of Section 3 of ASCE/SEI 4-16 (Reference 8.7). Section 5.1 describes the implementation of a one-step approach that addresses Section 3.1.2 of ASCE/SEI 4-16 for the BWRX-300 design that directly uses the results of the SSI analysis FE model as input for the design of the RB structural members.

Following the provisions of Section 5.1.5 of ASCE/SEI 4-16, the effects of structure-soil-structure interaction (SSSI) of R/B with surrounding foundations are incorporated in the design of RB SSCs as described in Section 5.3.7. Dynamic properties of subsystems, components, and equipment are included in the SSI analysis model based on the decoupling criteria of SRP 3.7.2 Subsection II.3.B, considering the effects of ESI as described in Section 5.3.6.

Per the requirements of ASCE/SEI 4-16, Section 5.1, the effects of non-vertically propagating seismic waves, soil separation, concrete cracking and soil secondary non-linearity on the seismic response and design of BWRX-300 RB are evaluated based on responses obtained from linear elastic and non-linear sensitivity SSI analyses described in Sections 5.3.3, 5.3.5, 5.3.8, 5.3.9, 5.3.10 and 5.3.11.

2.4 II/I Interaction Regulations

For the structures adjacent to the RB, the regulatory guidance of SRP 3.7.2 Subsection I.8, related to the requirements of interaction between Non-SC-I structures with SC-I SSCs structures that are referred to by the industry term "II/I interactions" is used. SRP 3.7.2 Subsection II.8 provides the following three II/I interaction criteria for which each non-SC-I structure should meet at least one:

A. The collapse of the non-SC-I SSC will not cause the non-SC-I SSC to strike a SC-I SSC.

B. The collapse of the non-SC-I SSC will not impair the integrity of SC-I SSCs, nor result in incapacitating injury to control room occupants.

C. The non-SC-I structure is analyzed and designed to prevent its failure under SSE conditions.

SRP 3.3.2, "Tornado Loadings," Revision 3, Subsection II.4 requires prevention of similar II/I interactions due to tornado loading so that failure of any structure or component not designed for tornado loads will not affect the capability of other SSCs to perform necessary safety functions. Because SC-I structures are designed for extreme wind conditions (tornado and/or hurricane), II/I interaction evaluations are performed for SSE as well as extreme wind loading.

As described in Section 6.0, the structural members of the CB, RwB and TB resisting horizontal loads are checked to ensure they can satisfy Criterion C so their collapse under extreme environmental design conditions, SSE and tornado and extreme wind loads, is prevented. The design also ensures that under these extreme environmental design conditions, the CB structure does not collapse to result in incapacitating injury to the control room occupants.

The II/I seismic interaction checks are performed considering limited inelastic responses in accordance to the provisions of ASCE/SEI 43-05 (Reference 8.4) and the governing design codes described in Sections 6.2 and 6.3, respectively.

2.5 Testing, Inspection and Monitoring Regulations

10 CFR 50 Appendix A, GDC 1, Quality standards and records, requires that important to safety structures be constructed and tested to quality standards commensurate with the importance of the safety functions to be performed. RG 1.142 and RG 1.136, "Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments," Revision 3, provide guidance for testing of safety-related nuclear concrete structures and concrete containments, respectively.

To meet inspection and testing requirements of 10 CFR 50 Appendix A, GDC 1, BWRX-300 construction inspection and testing programs satisfy:

- the geotechnical and foundation requirements of the NRC Inspection Manual 88131;
- the structural concrete activities requirements of the NRC Inspection Manual 88132; and
- the structural welding inspection requirements of NRC Inspection Manual 55100.

10 CFR 50.65, Requirements for monitoring the effectiveness of maintenance at nuclear power plants, specifies the maintenance rules for monitoring the performance or condition of structures against established goals to provide reasonable assurance that these structures can fulfill their intended safety functions. RG 1.160, "Monitoring the Effectiveness of Maintenance at Nuclear Power Plants," Revision 4, and NUMARC 93-01 "Industry Guideline for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants" (Reference 8.9) provide regulatory guidance for demonstrating compliance with the provisions of 10 CFR 50.65.

To meet the requirements of 10 CFR 50.65(a)(1), a monitoring program is established for a periodic assessment of the extent and rate of degradation of basemats and below grade exterior walls in accordance with the regulatory guidance of Section 1.3 of RG 1.160 and Sections 9.4.1.4 and 10.2.3 of NUMARC 93-01. Per Section 9.4.2.4 of NUMARC 93-01, the monitoring program can include non-destructive examinations, visual inspections, vibration, deflection, thickness, corrosion, or other monitoring methods.

Section 3.1 describes techniques for assessment of the site-specific soil and rock investigations following the guidance of RG 1.132 (Reference 8.64) and RG 1.138 (Reference 8.65).

Section 3.1.3 describes techniques to quantitatively characterize the geologic and engineering rock mass properties following the guidance of RG 1.132 and NUREG/CR-5738 (Reference 8.2).

Section 3.2 describes the construction inspection and monitoring program required during construction following the guidance of RG 1.132, including the requirements of NUREG/CR-5738 (Reference 8.2), Appendix A and B. During the BWRX-300 construction, a concrete compressive strength testing program is performed in accordance with RG 1.142 as described Section 3.2.2.

Section 3.3 describes the implementation of monitoring structures in accordance with 10 CFR 50.65, the guidance of RG 1.160 and NUMARC 93-01 for monitoring subsurface conditions to ensure any changes during the operational life of the BWRX-300 are bounded by the design.

a) ***Site Characterization***: comprehensive site investigation and laboratory testing programs are employed for the BWRX-300 that are beyond the recommendations in RG 1.132 (Reference 8.64) and RG 1.138 (Reference 8.65) to address specifics related to the design and construction of the deeply embedded RB. Section 3.1 provides requirements and recommendations for the BWRX-300 site investigations and subsurface material laboratory testing programs.

b) ***Excavation***: the removal of in-situ subgrade materials and dewatering of the excavation alters the initial stress and deformation site conditions that may influence the properties of the in-situ subgrade materials. The excavation may cause heave at the shaft bottom and displacements at the shaft sides that may result in changes of the properties and response of the soil and rock materials. The groundwater level and soil/rock movement are closely monitored in order to detect possible instabilities of subgrade materials during the excavation. As discussed in Section 4.3.4, comparisons between the results of FIA and monitoring program may be performed to assess, if the excavation behaves within the realm of expectations. Furthermore, the excavation stage provides the opportunity to inspect exposed surfaces of rock prior to construction. These inspections follow the guidance in RG 1.132 and Appendices A and B of NUREG/CR-5738, as discussed in Section 3.2.1.

c) ***Construction***: a successful construction process must be ensured because it may have a direct implication on the safety of the BWRX-300. The stability of the RB structure, foundation, and surrounding soil and rock are analyzed throughout the various construction stages by comparing data collected from the construction inspections and field observations, described in Section 3.2.2 and 3.4, respectively, with responses obtained from FIA, described in Section 4.0. Any changes in subgrade conditions during the construction process are evaluated and incorporated into the FIA models as discussed in Section 4.3.4.

d) ***Loading***: the behavior of the structure may be critical during loading stages that include the weight of fuel, water in the pools, and other permanent loads that were not previously introduced during construction. The field monitoring program and FIA modeling, described in Sections 3.4 and 4.3.4, continue through this stage and are used to confirm that the response of the subgrade and the RB structure to the additional permanent loads meets the safety requirements.

e) ***Start-up and Operation***: during this stage, two aspects are modeled and monitored by in-service and field monitoring programs described in Sections 3.3 and 3.4. The first is the continued monitoring of settlement and groundwater changes. Depending on soil types, long-term settlement response may be anticipated. Even if long-term settlement is not anticipated, this condition will require confirmation from monitoring. The second aspect relates to external events. Examples are forces from design ground motion, pressures and hazards from design flood, and potential subsurface deformation that originate from instabilities like undetected subsurface conditions or rock cavities, among others. As

described in Section 4.3.4, sensitivity studies may be performed to analyze potential formation of instabilities in the subsurface or to investigate the effects of flooding.

3.1 Site Investigation and Subsurface Material Testing Programs

The soil and rock properties and profiles for static and dynamic SSI analyses are established based on in depth site-specific investigations, field and laboratory testing programs described in Sections 3.1.1 and 3.1.2, following the guidance of RG 1.132 (Reference 8.64) and RG 1.138 (Reference 8.65). The intent of the guidance provided herein is to ensure adequate site investigation and subsurface material testing programs to yield the necessary inputs for the non-linear FIA described in Section 4.0, the probabilistic SRA described in Section 5.2.2, and the development of subgrade properties for the static and seismic design SSI analysis as described in Sections 5.2.1 and 5.2.4, respectively. Section 3.1.3 presents approaches for characterization of the rock mass properties based on the results of site investigation and testing programs.

Site characterization satisfies the guidelines presented in RG 1.132, including the guidance of NUREG/CR-5738, Appendices A and B. Beside the subgrade materials supporting the RB foundation, augmentation of the site investigation guidelines of RG 1.132, and specifically NUREG/CR-5738 Appendix B, are required for deeply embedded structures to provide a full characterization of the extent of in-situ materials surrounding the embedded RB shaft, including establishing an appropriate number, type, and extent of in-situ tests, such as borings, geophysical tests, and groundwater monitoring.

The quality and amount of site-retrieved data dictates the levels of epistemic uncertainty and aleatory variability that needs to be accounted for in the analysis and design of the BWRX-300. The design is therefore tied to a comprehensive site-specific investigation that follows the guidance of RG 1.132 and RG 1.138. The design of laboratory testing investigations depends on findings from the field activities and therefore a more in-depth discussion is omitted in this report.

Site investigation and subsurface material testing programs are recommended herein that are beyond the current regulatory guidance:

- address additional requirements specific to the innovative approaches implemented for the design and construction of the deeply embedded BWRX-300 RB in in-situ subgrade materials;
- ensure the design envelopes possible changes in the subsurface conditions during the excavation, construction, and operation of the BWRX-300 plant; and
- ensure the integrity of BWRX-300 structures are not compromised during the construction and operation.

The recommendations provided herein meet the minimum requirements for a generic greenfield candidate site. The actual number and types of field and laboratory tests are dictated by the site-specific conditions, such as the types of subgrade materials present at the site and their variation. For previously investigated sites, such as sites with issued Early Site Permits (ESP), the number of field and laboratory tests will likely be a subset of the recommendations provided in this report. At such locations, additional investigations and tests will be narrowed to close information gaps between the existing available information and specific needs for the siting of the BWRX-300.

3.1.1 Site Investigation Program

Figure 3-1 represents a preliminary layout of the BWRX-300 footprint and facilities with the deeply embedded RB being the only SC-I structure in the BWRX-300 plant. It is common practice to perform borings and tests below the footprint of the SC-I facilities and to deeper depths than the basemat (RG 1.132, Reference 8.64). The excavation approach minimizes the use of engineered backfill materials as well as the deployment depth of the BWRX-300 RB and requires a subsurface investigation that covers areas beyond its foundation perimeter.

When bedrock units are anticipated to be encountered at depths for engineering purposes, geologic mapping of outcrops should be completed prior to finalizing the number, orientations, and locations of the field investigation borings and tests. This geologic mapping is intended to characterize the anticipated rock mass, discontinuities and to allow for modification of the field investigation to collect appropriate data near the RB shaft.

The diameter of the RB SC-I footprint is relatively small when compared to footprints of typical conventional nuclear plants. The characterization of a small portion of the subsurface environment would be insufficient to adequately characterize the variations and uncertainties in the site subsurface conditions and provide inputs for the Approach 3 probabilistic SRA described in Section 5.2.2. Tests, such as seismic refraction or reflection studies that are useful to map bedrock or detect potential voids become meaningful and possible only when covering greater areas. Measurements of shear-wave velocities (V_S) and compression-wave velocities (V_P) are not sufficient to characterize lateral variability if these are made just a few meters apart.

In order to address the specific requirements of the BWRX-300 RB design, the subsurface site investigations are performed following the guidelines of RG 1.132 for SC-I type site investigations considering the combined footprint areas of the RB SC-I foundation and the adjacent TB, CB and RwB foundations. The extended area considered by the BWRX-300 subsurface site investigation ensures an adequate characterization of the subsurface conditions under the TB, RwB and CB foundations and resulting surcharge loads, which are important for the design of the deeply embedded RB structure and seismic design of RB SC-I SSCs.

Appendix D of RG 1.132, Spacing and Depth of Subsurface Explorations for Safety-Related Foundations, specifies the need for at least one boring underneath each projected safety-related structure or 1 boring for each 900 m^2. The footprint of the main containment shaft and the above ground surrounding structures is about 1 Ha (10,000 m^2). This implies that at least 10 borings would be required for the site investigation. RG 1.132 indicates that the boring depth should go past "the maximum required depth for engineering purposes." If bedrock is encountered, then the boring should penetrate past zones of weakness that could affect foundation performance and extend at least 6 m into sound rock. For the BWRX-300, the maximum required depth for engineering purposes d_{max} is set at approximately 120 m, a depth that is the greater than the following:

a) The depth of the shaft plus twice the diameter of the shaft, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition, and

b) Twice the width of the plant's footprint, which corresponds to a zone where the change of vertical stress is expected to be less than 10 % from the in-situ condition.

Table 3-1: Site Investigation for the BWRX-300

Test Type		Test Purpose	Number of Tests [1]
1	Geotechnical borings	− Measure Standard Penetration (SPT) − Measure Cone Penetration Resistance − Sample soils and rock for visual classification and laboratory testing − Rock Quality Designation (RQD) − Characterize rock mass and discontinuities − Perform pressuremeter tests on weak to moderately soft rock portions to have data parameter for estimation of elastic moduli − Measure in-situ stress (overcoring, hydraulic fracturing)	− 3 borings at perimeter and center of containment down to 120 m − 2 borings at perimeter of containment down to a depth of 60 m − About 18 additional borings designed to cover the footprint of the main facilities, meet the regulatory guidance, and characterize the subsurface as a unit. (see Figure 4-1)
2	Wells	− Groundwater characterization (pump and slug tests, baseline groundwater quality) − Characterize groundwater flow direction and quantify hydraulic gradients	− 9 wells at the center and edge of containment to anticipated depth of 60 m − 4 wells down to a depth of 60 m covering the footprint of the facility
3	Geophysical boring	− Measure V_P and V_S with at least two methods: seismic downhole survey, crosshole, and/or and PS Log suspension survey.	− One boring down to 120 m at center − 4 borings at perimeter of containment down − 4 borings located a distance apart from RB to allow for wider cross sections and correlations to refraction or reflection surveys
4	Refraction Survey	− For sites in which a bedrock horizon is identified by the boring program, perform seismic refraction to obtain a three-dimensional mapping of the bedrock horizon and the thickness of weathered layers	− One grid of surveys covering the footprint extension of the facility
5	Seismic reflection survey	− Identify if voids, sinkholes, karst, or faults are present beneath the footprint of the facilities	− Three longitudinal and two to three transverse reflection sections
6	Borehole Televiewer (Optical/Acoustic)	− Observe rock surface directly, subsurface lithology and structural features such as fractures, fracture infillings, foliation, and bedding planes. − Measure orientation and spacing of rock discontinuities − Packer water-pressure tests in rock − Measure in-situ stress (overcoring, hydraulic fracturing)	− Relevant for rock conditions, over which boring recovery and RQD allow for an open borehole. − The proposed 8 televiewer locations will support a better characterization of the rock mass and as a substitute for potential inspection limitations due to the construction process.

[1] Number may be adjusted depending on encountered site conditions and site available information

Table 3-2: Anticipated Boring Program

Boring	Depth (1) (m)	SPT(2)/ CPT(3) Coring	VEL DH(1)	VEL PS LOG(2)	Well
B-01	120	✓	✓	✓	✓
B-02	120	✓	✓	✓	✓
B-03	120	✓	✓	✓	✓
B-04	60	✓			✓
B-05	60	✓			✓
B-06	60	✓			
B-07	30	✓			
B-08	60	✓			
B-09	80	✓	✓	✓	
B-10	80	✓	✓	✓	
B-11	60	✓			
B-12	60	✓			
B-13	80	✓			✓
B-14	80	✓	✓	✓	
B-15	60	✓			
B-16	80	✓			✓
B-17	60	✓			✓
B-18	80	✓	✓	✓	
B-19	60	✓			✓
B-20	60	✓			
B-21	100	✓			
TOTAL	~1600	~21	~7	~7	~9

Notes:
(1) Subject to change based on site conditions
(2) SPT: Standard Penetration Test
(3) CPT: Cone Penetrometer Test
(4) VEL DH: Downhole velocity (VEL) test
(5) VEL PS Log: PS Suspension log velocity (VEL) test

3.1.2 Laboratory Testing Program

A laboratory testing program is performed on soil and rock samples collected from the site investigation program in accordance with the regulatory guidance of RG 1.138 (Reference 8.65) to obtain data for the analysis and design of the BWRX-300 RB. The scope and extent of the BWRX-300 laboratory testing program address the specific requirements of deeply embedded BWRX-300 design that requires a reliable set of data from laboratory tests for developing geotechnical inputs characterizing the properties of each subgrade material present at the site.

A laboratory testing program is implemented that depends on the site-specific subsurface conditions, the specific analysis requirements, and the need for sufficient data to adequately characterize variations in subsurface material properties. A sufficient number of laboratory tests are performed to minimize the uncertainties in the design related to these geotechnical input

parameters by providing reliable estimates for the statistical parameters (mean and standard deviation values) of the measured material properties. The systematic (bias) errors are minimized by a carefully executed equipment calibration and sample management programs. Estimates of measuring bias are developed based on comparisons of measurements of physical parameters obtained from different types of subsurface material property tests.

Testing to estimate strength parameters for appropriate rock discontinuities in bedrock units should be completed using appropriate methods that may include direct shear test (References 8.66, 8.67), triaxial strength tests (Reference 8.68), and appropriate methods identified in RG 1.138 (Reference 8.65). This testing shall determine the strength parameters (e.g., peak friction angle, residual friction angle, and apparent cohesion) of discontinuities and similar weak planes in rock. Testing of artificial interfaces may be completed to determine the strength properties at the interfaces with the RB structures.

At a minimum, the laboratory tests of soil materials include:

- Index testing (classification, weight, plasticity, grain size)
- Strength testing (shear tests, triaxial tests)
- Deformability tests (triaxial tests, consolidation tests)
- Permeability
- Chemical testing (chlorides, sulfates, pH, Resistivity)
- Dynamic tests (Resonant Column Torsional Shear (RCTS), cyclic triaxial)

The minimum laboratory tests required to develop properties for rock materials include:

- Uniaxial Compressive (UC) strength,
- Triaxial compressive strength and elastic moduli,
- Direct shear tests,
- Petrography,
- Dynamic tests (sonic pulse wave velocity, Free-Free Resonant Column velocity tests)

Other tests, such as the expansion, creep, mineralogy, erodibility, durability, X-ray diffraction tests may be performed on an as-needed basis.

3.1.3 Characterization of Rock Mass Properties

The properties of rock are characterized based on the information collected from the site investigation and laboratory testing programs described in Sections 3.1.1 and 3.1.2. Rock joints, bedding planes, discontinuities fracture and other weak zones are evaluated to determine:

- the type of temporary excavation support and improvements required during construction;
- groundwater conditions and required seepage control measures; and
- possible effects on the rock pressure loads on the RB shaft.

The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones may affect methods used to excavate rock for construction of the shaft. Methods that are used to compensate for these weak zones include:

- over-excavation and backfilling;
- internal structural support;
- spot or pattern rock reinforcement (i.e., rock bolts or anchors); and
- surface treatments (i.e., mesh, straps, shotcrete).

Additionally, the existing groundwater conditions and the potential control of seepage through cavities, fracture zones, joints, bedding planes, and discontinuities is considered. Seepage control may include slurry walls, grouting prior to excavation, grouting during the excavation, freezing, drains, dewatering wells, sumps and other methods. The existing groundwater conditions and appropriate modifications to the rock mass classification, consistent with the selected method, shall be determined as part of the Site Investigation Program in Section 3.1.1.

The in-situ state of stress in the bedrock shall be evaluated. This process shall include reviewing the state of stress in the crust as part of evaluating the tectonic framework and unrelieved stresses in bedrock near the site. A review of regional and/or local references that evaluate the current state of stress in the crust and the potential for horizontal stresses from tectonic activity, residual strains, or topographic conditions shall be used to assess the likelihood for increased horizontal stress in the bedrock. Based on the results of this review, in-situ tests like those shown in Table 3-1 may be considered to make site-specific measurements of the in-situ state of stress in bedrock formations as part of the geotechnical borings and borehole televiewer tests. All potential and/or appropriate tests for measuring in-situ stress are not identified in this document because the appropriate tests will be specific to each site.

Discontinuities and other zones of weakness within the rock mass may also control the stability of individual blocks or the rock mass when the orientation is disadvantageous and/or the spacing of discontinuities is sufficiently dense. The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. These discontinuities or weak zones may form a system of blocks or wedges where strength within the individual blocks is high, but strength along the weak zones between the blocks is highly anisotropic.

To adequately assess and consider weak zones in rock masses, RG 1.132 (Reference 8.64) and NUREG/CR-5738 (Reference 8.2) provide guidance on geologic mapping, logging and characterizing of rock materials. Geologic mapping and geotechnical borings described in Section 3.1.1 are used to characterize the intact rock, rock discontinuities, and the rock mass. Frequently, optical and acoustic televiewers (OTV/ATV) are used in conjunction with geologic mapping and oriented or classical rock coring methods to map the depths, orientations, aperture, and other characteristics of the discontinuities. The type of information and testing required for the rock mass will depend on the specific subgrade conditions as well as the rock mass classification selected for the site. When other data or geologic mapping indicates near vertical discontinuities may be present, inclined borings may be used to properly characterize the orientation and strength of near vertical discontinuities.

where: $$\text{JCond}_{89} = 35\frac{\left(\frac{\text{Jr}}{\text{Ja}}\right)}{\left(1+\frac{\text{Jr}}{\text{Ja}}\right)}$$

As described in RG 1.132, characterization of the shear strength for planar discontinuities, such as bedding planes, faults, fracture zones, joints, and shear zones typically include laboratory testing of subsurface discontinuities recovered from samples (e.g., direct shear and triaxial compressive strength tests) or, less commonly, in-situ tests of the discontinuities under specific loading conditions. Because the most common method is testing recovered subsurface samples, empirical corrections are required for surface roughness, intact surface strength, and the scale of the tested sample (e.g., Barton-Bandis criterion).

When the rock discontinuities are filled with another material, the shear strength may decrease or increase depending on the type of infill material. Testing of the infill material is required when there is a significant thickness of weaker material that may control the strength of the discontinuity. When a nonlinear relationship between shear strength and normal stress (e.g., Barton-Bandis criterion) is not desired, the equivalent friction angle and cohesion may be determined from the tangent to the nonlinear relationship for the shear strength of planar discontinuities.

Cavities in the rock mass from karst or dissolution may decrease the effective rock mass modulus and create a highly variable interface between the rock and overburden. The presence of cavities should be identified during the subsurface investigation. Consistent with RG 1.132, the spacing and depth of investigation locations should be reduced to detect the anticipated features.

A grouting program may be required to fill cavities and control seepage. The grouting program should include the potential to remove infilling from cavities using a water wash and fill the cavities as much as possible with grout. Replacing infill or open cavities with grout should increase and control variations in the rock mass modulus around and beneath the structures. Contact grouting is also required after construction of the shaft to avoid irregular external loading from voids – natural or due to overbreak during construction – on the exterior of the shaft. The rock surface may require modification through excavation or ground improvement to avoid significantly different stiffness along the shaft. Epikarst may form pinnacles or similar features that may result in variable stiffness along the shaft near the bedrock and overburden interface. The effect of potential cavities in the rock mass and variations at the bedrock and overburden interface on shaft deformation are evaluated on a site-by-site basis.

3.2 Construction Inspection and Testing Program

3.2.1 Excavation and Foundation Inspections and Testing

Excavation and foundation inspections and testing programs are implemented for the BWRX-300 that meet the geotechnical and foundation requirements of the NRC Inspection Manual 88131 (Reference 8.15), including:

A. Key Site Parameters are verified by checking if the required values for average allowable static bearing capacity and maximum allowable dynamic bearing capacity for normal plus SSE loading have been met at the excavation depth.

B. Soundness of the exposed rock is checked by qualified personnel to confirm the results of rock mass characterization described in Section 3.1.3. This includes visual inspection and testing of:

- Rock material properties, such as rock type, color, particle size, hardness, and strength.
- Rock mass properties, such as rock structure, shear strength, deformation modulus, hydraulic conductivity, and attitude.

3.2.2 Building Structure Construction Inspections and Testing

The BWRX-300 RB construction inspection and testing program satisfy the structural concrete activities requirements of the NRC Inspection Manual 88132 (Reference 8.16) and structural welding inspection requirements of NRC Inspection Manual 55100 (Reference 8.17). The program includes:

- The visual surface inspection acceptance criteria that include quantitative limits for the appearance of leaching or chemical attack, pop outs or surface voids, scaling, spalling, corrosion staining, settlements, and cracks.
- ACI 349.3R guidance (Reference 8.18), which is recommended by ASME XI Rules for Inservice Inspection of NPP Components, Subsection IWL for visual inspections of exposed surfaces. ACI 349.3R requires that accessible concrete surfaces do not have voids greater than 2 inches; scaling is limited to 8 inches in diameter and 0.75 inches in depth; and cracks are limited to widths of 0.04 inches or smaller.
- ASME XI, Subsection IWL 1220 (b) and (d) exempts concrete surfaces that are covered by a liner or adjacent to a foundation or backfill from detailed visual inspections.
- Concrete surfaces exposed to soil, backfill, or groundwater are evaluated to determine susceptibility of the concrete to deterioration and the ability to perform the intended design function under conditions anticipated until the structure no longer is required to fulfil its intended design function. The evaluation includes the following:

  a) Existing subgrade conditions, including groundwater presence, chemistry, and dynamics; aggressive below-grade environment, or other plant-specific conditions that could cause accelerated aging and degradation.

  b) Existing or potential concrete degradation mechanisms, including, but not limited to, aggressive chemical attack, erosion and cavitation, corrosion of embedded steel, freeze-thaw, settlement, leaching of calcium hydroxide, reaction with aggregates, increase in permeability or porosity, and combined effects.

  c) Design and construction criteria associated with the inaccessible concrete, including structural design, detail and reinforcement, design recommendations implemented with regard to environmental exposure conditions, materials used, mixture proportioning, concrete production and placement, design and construction codes used, conformance of the structure to original design and performance of any reanalysis.

Table 3-4: Degradation Conditions and Criteria for Accessible Steel Structures

Degradation Condition	First-Tier Criteria	Second-Tier Criteria
Corrosion and/or corrosion stains	Absence of condition(1) (2)	Condition present, but determined acceptable after further review(3) (4) (5)
Bulges or depressions in liner plate	Absence of condition(1)	Condition present, but determined acceptable after further review(3)
Cracking/degradation of base or weld metal	Absence of condition(1)	Condition present, but determined acceptable after further review(3)
Leakage/Seepage (presence of water)	Absence of condition(1)	Condition present, but within original design limits of active leak-detection system and the leaking material and source do not present any adverse consequences(3)
Detached embedments or loose bolts	Absence of condition(2)	Condition present, but determined acceptable after further review (4)

(1) Section 5.1.2 of ACI 349.3R (Reference 8.18)

(2) Section 5.1.3 of ACI 349.3R (Reference 8.18)

(3) Section 5.2.2 of ACI 349.3R (Reference 8.18)

(4) Section 5.2.3 of ACI 349.3R (Reference 8.18)

(5) Section IWE-3500 of ASME XI (Reference 8.20) provides a threshold of 10% loss of nominal wall thickness.

3.4 Field Instrumentation Plan

Field instrumentation that beyond the current regulatory guidelines, is deployed to monitor the magnitude and distribution of pore pressure and amount of deformation during excavation, construction, loading and continuing through the BWRX-300 plant operation. The instrumentation provides recordings that can frequently be benchmarked against design estimates. Short-term and long-term settlement monitoring plans are developed that can detect both vertical and horizontal movements in and around the structures, as well as differential distortion across the foundation footprint and differential settlements between the CB, TB, RwB and RB foundations.

The specific locations of the sensors inside and outside of the RB shaft are dictated by the subsurface conditions and areas identified in the design where maximum stress, strain, and pore pressures are anticipated along the perimeter of the shaft. The definitive number of instruments is established during design stages of the monitoring system considering that the field instrumentation system shall be capable of:

A. Measuring the rate of heave during excavation, especially at the end of excavation and at the bottom center and edges of the shaft.

B. Measuring the rate of lateral displacement of excavation walls, throughout its depth, during and at end of excavation.

C. Measuring the distribution of pore pressures around and below the RB shaft.

D. Measuring the total settlement and tilt of the RB shaft, during construction, loading, and operation; this will require deploying a system of sensors and survey monuments throughout the perimeter of the shaft at bottom, medium depth, and plant grade.

E. Measuring settlement of the auxiliary and surrounding structures of the BWRX-300.

Figure 3-3 indicates the required implementation period that the field instrumentation has to accommodate. Some instruments will be temporary while others are permanent. Some instruments, such as piezometers, are installed prior to excavation. Installation for extensometers or other survey monuments are to be taken at the appropriate stage of the BWRX-300 life.

To achieve the required monitoring capabilities, the field instrumentation consists of four primary elements:

1. Piezometers to measure pore pressure distribution. Vibrating wire piezometers are preferred for this purpose as they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis. Piezometers should be screened at elevations that are representative of the site-specific hydrogeologic conditions.
2. Settlement monuments placed directly on concrete, preferably on the corners of the structures at grade that are accessible with conventional surveying equipment.
3. Settlement sensors and extensometers used for settlement prone soils or deformation prone rock masses.
4. Earth pressure sensors to monitor vertical and lateral pressure along the walls of the shaft.

For deployment in soft soil conditions, settlement sensors are installed within a borehole attached by a Borros anchor as described in Reference 8.21. For hard soil and rock conditions, sensors may consist of rod type extensometers anchored below loading points. The borehole extensometer includes anchors, extension rods and a reference head. The anchor is connected to the head of the instrument by extension rods typically placed within a protective sleeve. This sleeve ensures that the rods can move freely and translate all movement of the anchor to the tip of the rod. The movement of the rock or soil mass relative to the head can then be calculated by measuring the displacement of the tip of the extension rod to a reference plate located in the head of the extensometer as the one described in Reference 8.22. The instrument can be used to measure deformation of laterally loaded retention walls and to monitor settlement in foundations.

The groundwater levels at the site are monitored using pressure transducers installed in multiple screened wells installed across the site. This data provides groundwater elevations, groundwater flow direction(s) and groundwater gradients. This information is used during excavation and construction for estimating seepage rate, short-term dewatering rates, and effective stresses under static and dynamic conditions.

When practical and applicable, sensors are connected to a datalogger(s) programmed to read the sensors periodically. Some of these sensors are installed in cased boreholes and the sensors can be removed, maintained, or replaced during the needed phases of the project. Other sensors, such as the earth pressure sensor, need to be buried in the subsurface and cannot be removed or replaced once backfilled. Such sensors are installed with redundancy to monitor the necessary data for the specific duration of the project phase when such data is used.

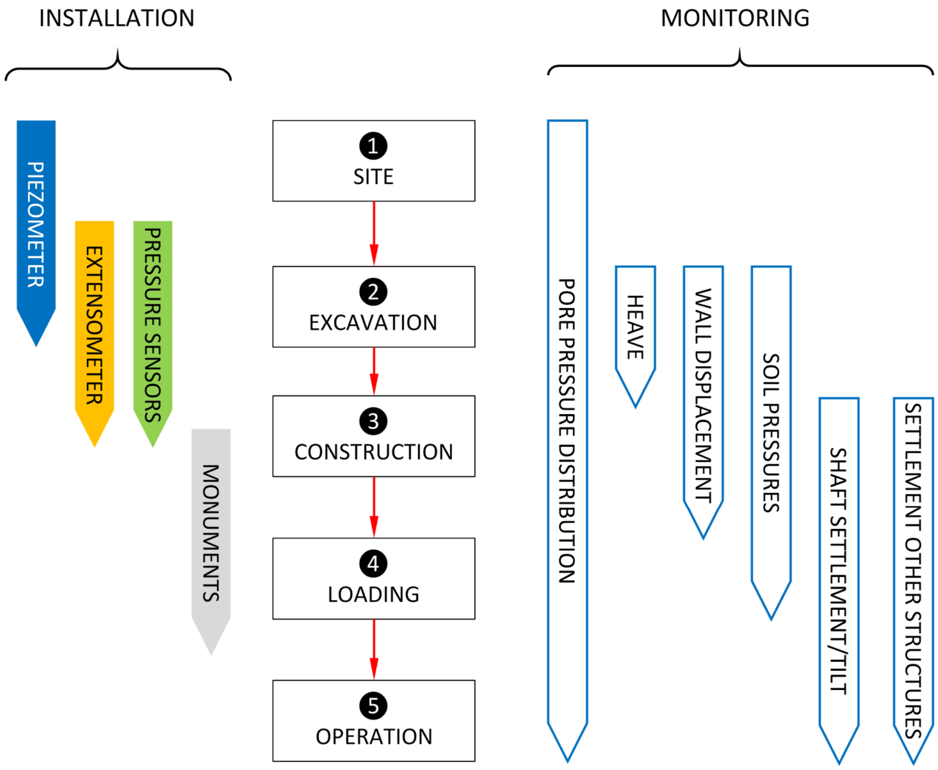

Figure 3-3: Field Instrumentation and Monitoring

3.5 Summary of Investigations, Testing, Inspection and Monitoring Programs

The following BWRX-300 requirements and innovative approaches that are related to the site investigations, testing, inspection, and monitoring programs, presented in this section of the report, may be referenced during future licensing activities:

(1) The site investigation program requirements provided in Section 3.1.1 include recommendations for the minimum number of borings and field tests, types of field tests, boring locations and depths that are beyond the current guidance of RG 1.132 (Reference 8.64).

(2) The laboratory testing program approach presented in Section 3.1.2 includes the minimum requirements for laboratory testing of soil and rock materials that are beyond the current guidance of RG 1.138 (Reference 8.65).

(3) The approach for characterization of rock mass properties presented in Section 3.1.3 include examples of empirical engineering and geomechanical rock mass classifications to quantitatively characterize the geologic and engineering parameters of rock masses and recommendations for characterization of the shear strength of planar discontinuities that are beyond the current guidance of RG 1.132 and RG 1.138.

(4) The construction inspection and testing program approach presented in Section 3.2, include requirements for the minimum frequency of concrete compressive strength tests,

4.0 FOUNDATION INTERFACE ANALYSIS

The purpose of a FIA is to ensure that BWRX-300 RB, CB, TB and RwB structures and supporting media, soil and/or rock, meet the stability requirements and regulatory guidance of SRP 2.5.4, with emphasis in Subsections 2.5.4.3, 2.5.4.5, 2.5.4.6, and 2.5.4.10. The construction plans, including possible ground improvements, excavation support and foundation interface design are evaluated based on the results of FIA at different life stages of the BWRX-300 using non-linear models that have the capability of sequencing construction and loading and reproducing the stress and deformation fields. The results of the FIA are also used for verification of the RB shaft design as described in Section 5.1.3.

The scope and extent of BWRX-300 FIA are beyond the current regulatory guidance and address specifics related to the design and construction of deeply embedded SMRs identified in NUREG/CR-7193, Sections 1.5.10 and 1.5.11 (Reference 8.1). The predicted foundation interface behavior is compared against physical observations from the monitoring programs described in Sections 3.2, 3.3, and 3.4 to:

- allow for confirmation of the analyzed stability conditions;
- assess the effects of excavation and construction on the properties of in-situ subgrade materials;
- evaluate the effects of new loads or changes in the site conditions that may occur during the operation life of the BWRX-300 plant; and
- evaluate potential subsurface deformations that may originate from subsurface instabilities.

The implemented approach offers assurance that the actual observations are within expected ranges that have been anticipated during the design and prior to the construction.

4.1 Foundation Interface Analysis Model

The BWRX-300 stability is monitored throughout the remainder of its life stages (excavation, construction, loading, and operation) by implementing a benchmark process that provides a link between the expected and measured response of the system. The process involves development of a numerical model that examines the response that the BWRX-300 and its surrounding media exhibits due to alterations of in-situ subgrade conditions. Such responses are monitored, both through FIA model response and field measurements. The numerical model is calibrated using the field measurements to predict future response of the structure. This process represents a tool to reassure structural and site responses stay within the design bounds.

The FIA numerical model has the following features:

A. Three dimensional

B. Capability to incorporate non-linearity in the stress-strain behavior of soil and rock; this feature addresses the non-linear behavior at low and high strain, and even cases where physical instabilities may be present. The non-linearities in stress-strain of soils and rocks is captured by the use of constitutive models described in Section 4.2 that best fit the subgrade materials of the deployment site. These constitutive models range from the simplest elastoplastic

Mohr-Coulomb to other more sophisticated cases that incorporate strain-hardening/softening, strain dependent elastic and shear moduli, or rock failure criteria such as Hoek-Brown.

C. Interface modeling described in Section 4.3.1; allows the introduction of the response and failure criteria between geometric zones; this feature is necessary to analyze faults, rock slip surfaces, or other discontinuities around the structure. The interface modeling has non-linear modeling capabilities.

D. Interface modeling between soil/rock and structure described in Section 4.3.1; which is necessary to incorporate interaction between concrete and soil/rock via friction, accounting for the selected construction method and final configurations at the structure-soil/rock contacts. Non-linear behavior and separation are parts of the capability of this feature.

E. Structure modeling, which may be limited to the main civil/structural components of the RB: main walls, floors, pools, and auxiliary structures.

F. Soil/rock anchors and geogrids, which are used to simulate stabilization of the excavation and any associated potential failure surfaces.

G. Fluid-soil interaction, which may be considered if the modeling the position of a static, horizontal groundwater table is not sufficient for the complexities in the design and construction of the BWRX-300 RB. Pore pressures are dependent on the permeability of the subsurface media, the hydrogeologic configuration, and the dewatering strategies for construction and operation.

H. Staging analysis with time-dependent capabilities, which enables modeling the interaction of the structure and surrounding subgrade from excavation, through construction, loading and final operation. The model is capable of following stress/strain response as stress regimen changes from unloading during excavation to reloading after construction and during operation.

4.2 Subgrade Material Constitutive Models

Constitutive models define the relationship between the stresses and strains for different materials. Non-linear constitutive models are used for soils, rocks, and interfaces, or a combination of them.

The selection of the non-linear constitutive models for the BWRX-300 FIA is based on site-specific characteristics of the subsurface materials and the expected stress levels that result from dewatering, excavation, and loading. Regardless of the selected constitutive approach, the numerical model handles the potential for development of plastic zones or interfaces that can result from planes of weakness, presence of voids or cavities, or simple excess loading.

The parameters defining the soil and rock constitutive models are developed based on data obtained from the field and laboratory testing programs described in Section 3.1 and calibrated based on data collected from the field instrumentation program described in Section 3.4. This calibration includes modifying select input parameters for the soil and rock constitutive models or the interface models to better match the data collected from the field instrumentation program.

4.2.1 Soil Constitutive Models

Non-linear constitutive models are applied to soil materials. The Mohr-Coulomb failure criterion is typically used to represent shear failure in soil. Soils with Mohr-Coulomb behavior experience

4.3 Non-Linear Foundation Interface Analysis Approach

The FIA addresses the following aspects:

A. Interface modeling, described in Section 4.3.1, including both (a) contacts between structure and soil/rock, and (b) fault or joint planes or interfaces between bedding units in a geologic formation.

B. Structural modeling of the main civil/structural components of the BWRX-300 and auxiliary facilities, described in Section 4.3.2, along with varying live and dead loads throughout the construction process.

C. Fluid Soil Interaction, described in Section 4.3.3, to capture an adequate distribution of the space and time variation of pore pressures.

D. BWRX-300 life stages: siting, excavation, construction, loading, and operation described in Section 4.3.4.

4.3.1 Interface Models

4.3.1.1 Interfaces Between the Structures and the Subgrade Media

The behavior of the contact at the base might not be critical for the RB because sliding and overturning are likely controlled by the deep embedment. However, the behavior of contact between the walls and soil, influences the soil pressures exerted on the structure along its embedded depth. The contact behavior depends on the selected construction methodology and changes through construction. For example, the contact condition of the BWRX-300 RB outer wall, when poured using a slurry wall or rock face as formwork, is different than the contact gained from a typical construction and backfill/grouting process. Figure 4-1 provides a schematic showing interfaces between structure and the surrounding media.

The interface is modeled, as is the case for the soil, with the use of an elastoplastic relationship based on an elastic deformation modulus and shear resistance. Figure 4-2 shows an example of interface rheologic modeling typically used for BWRX-300 FIA. A series of spring couplers are simulated at the connecting grid points at the interface. Each spring is represented by an elastoplastic model with Mohr-Coulomb criterion for shear failure.

When interface elements are used to represent the structure and soil/rock interaction, node pairs are created at the interface. From a node pair, one node belongs to the structure and the other node belongs to the soil/rock. The relative displacements (i.e. slipping/gap opening) can be simulated through elastic-perfectly plastic springs between these two nodes. Typically, two sets of springs are used for interface elements. One elastic-perfectly plastic spring to model the gap displacement and one elastic-perfectly plastic spring to model slip displacement. The simulation of gaps opening between the structure and soil/rock can be achieved through activating a tension cut-off for the spring that does not allow any tension at the interface.

The parameters of the slipping spring can be taken from the material set of the adjacent soil/rock elements or strength tests on natural and artificial discontinuities from the site investigation, laboratory testing program and characterization programs as described in Sections 3.1.2 and 3.1.3. The development of the interface parameters should be consistent with the limitations and modeling guidance of the software and interface model used for the nonlinear FIA. A strength

reduction factor can be used to adjust the spring stiffness based on the roughness of interaction and soil/rock residual strength when the sliding occurs. It is also possible to assign strength properties to interface elements based on direct measurements. If planar geosynthetic products are used during construction of the wall, shear properties are assigned to the interface elements representative of shear properties at geosynthetic/soil interfaces.

As is the case for soil and rock material constitutive models, the use of complex modeling capabilities for modeling interfaces introduces the challenge of identifying adequate input physical parameters. To address the uncertainties in these input parameters, sensitivity analyses may be conducted by adjusting spring stiffness and shear strength directly or through strength reduction factors. These types of analyses provide insight to understand the uncertainty introduced by interfaces in the stress distribution and deformation response of the structure.

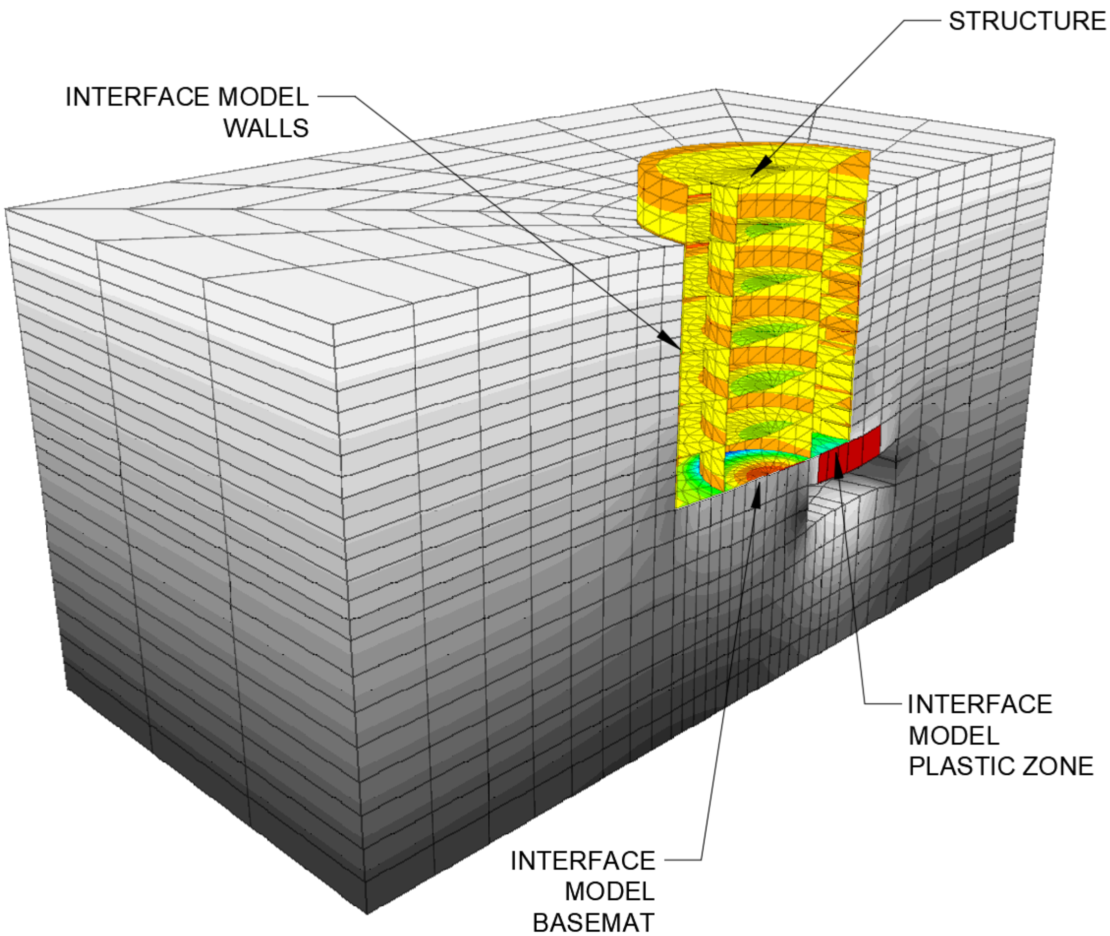

Figure 4-1: Location of Interfaces between Soil and Structure

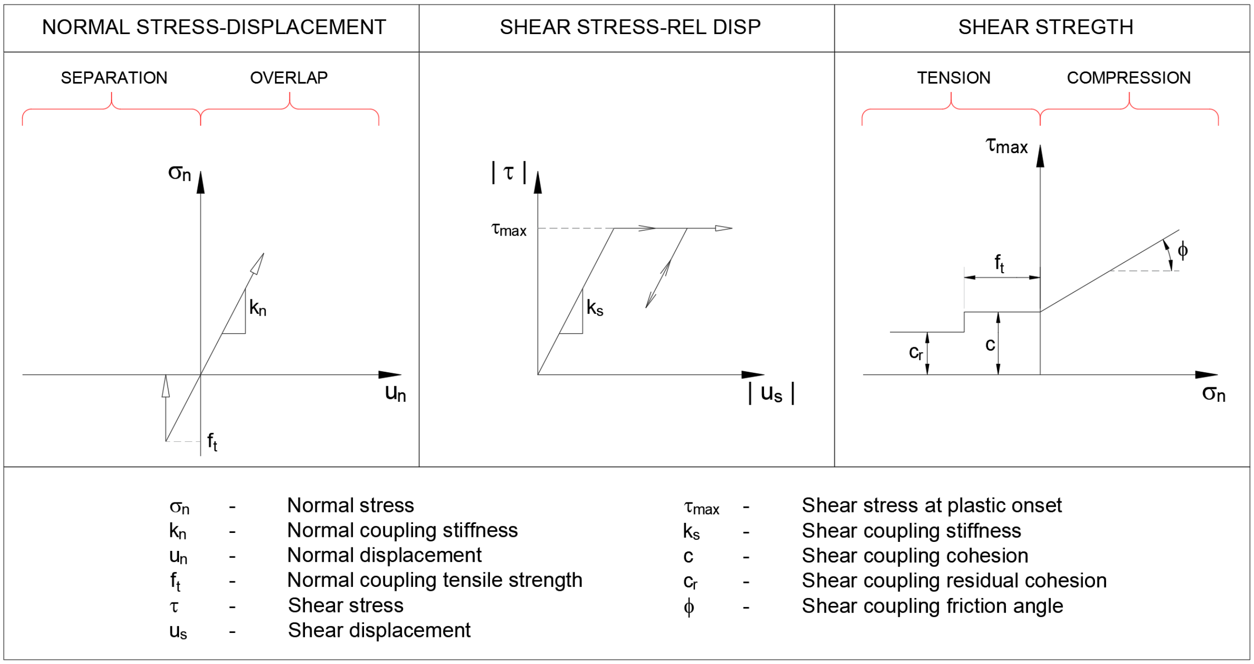

Figure 4-2: Interface Rheologic Modeling

4.3.1.2 Fault or Joint Planes or Interfaces Between Bedding Units in a Geologic Formation

The embedment depth of the BWRX-300 allows the possibility that soil rock interfaces, bedding interfaces, and other joints (Figure 4-3) may be in contact with the sides and base of the structure. These features may have planar or irregular configuration, and may be horizontal or with dipping, and even striking angles with respect to the position of the structure. The non-linearity and behavior of the joints are analyzed throughout the life stages of the reactor. These interfaces are modeled using similar interface modeling approaches as described in Section 4.3.1.1. The strength properties assigned to the interface elements along a rock discontinuity, i.e. bedding, are obtained from laboratory testing data described in Section 3.1.2. When multiple strength tests are performed for rock discontinuities, the weakest strength parameters can be used for the interface elements or sensitivity analyses may be completed similar to Section 4.3.1.1. Strength reduction factors may be used to adjust the spring stiffness and shear strength based on the strength of the interface where the sliding occurs.

operation. The model can simulate short-term as well as long-term dewatering or pumping as dictated by field conditions. The model simulates the changes in pore water pressures of the soil in response to unloading during the excavation stage and loading during construction and loading stages.

4.3.4 Analysis Staging Approach

Section 3.2 provides a description of the life stages of the BWRX-300, starting from the site investigation and ending with the plant operation. The BWRX-300 FIA are performed on numerical models that have the features to perform an integrated analysis of the stress, and deformation fields for each of the identified life stages:

4.3.4.1 Site Characterization

The FIA begins with the site itself, in its native condition, prior to any excavation or construction activities. During this stage, the initial stress conditions are aligned with the initial baseline displacement field. Initial stress conditions include, if applicable, the influence of groundwater aquifers and measured horizontal stresses.

4.3.4.2 Excavation

During the BWRX-300 RB shaft excavation, shown on Figure 4-4, soils and rock around and below the shaft may experience tensile stresses. The selected constitutive models allow for expansion response of soils resulting in heave or added pressures on excavation support structures. The changes in site conditions made prior or during the excavation are introduced in the FIA model following the sequence of the excavation plan. Non-linear interfaces are modeled between stabilization walls and soil.

As shown on Figure 4-5, the excavation simulation resembles the scheme planned for the specific site, by staging the removal of soil layers as excavation progresses and excavation support and site improvements are made. The stability of the excavation is verified in analytical space and later compared against field observations. The process allows for the design and monitoring of a safe excavation.

At the end excavation, the stress and displacement fields of the surrounding media, as well as the distribution of pore pressure, will have evolved. The "after excavation" condition is used as the initial condition for the analysis of the construction stage.

- groundwater hydrostatic pressure; and
- overburden loads and the interaction with the surrounding RwB, CB and TB foundations and structures.

Furthermore, the interaction with the surrounding subgrade determines the boundary conditions at the RB below-grade shaft exterior wall and basemat interfaces thus affecting the structural response and stress distribution from other static and dynamic loads such as operating and accidental thermal and pressure loads.

In order to adequately account for the SSI effects, the one-step approach, as defined in Section 3.1.2 of ASCE/SEI 4-16 (Reference 8.7), is implemented for the design of the BWRX-300 RB structure using a linear elastic SASSI (a system for analyses of soil-structure interaction) analysis approach described in Section 5.3. Static and dynamic structural stress demands are obtained directly from the results of SSI analyses of combined models that include FE representations of the RB structure and the surrounding soil. The surrounding subgrade is represented by layered half-space continuum with equivalent linear elastic stiffness properties and complex damping.

Stress demands on the RB structural members due to static earth pressure, structural self-weight, equipment weight and life loads are calculated by applying 1-g gravity loads on the combined model of the RB structure and the subgrade continuum. The structural demands due to overburden pressures from the nearby foundations are also calculated by the 1-g static analysis. Additional static analyses are performed to calculate the structural demands due to hydrostatic wall pressures from the pool water, normal operating and accidental pressure loads. Separate analyses provide the structural demands due to normal operating and accidental pressure and thermal loads. Structural demands due to seismic inertia loads and dynamic soil pressure loads are obtained from seismic SSI analyses that are described in Section 5.3.

The methodology used for development of RB FE model is based on the methodology described in Section 5.1.1 and the SSI modeling assumptions presented in Section 5.1.2. Equivalent linear properties are used as input for the static and seismic SSI analyses developed as described in Sections 5.2.1 and 5.2.4, respectively. Section 5.1.3 presents the unique BWRX-300 approach used to demonstrate that the linear-elastic SSI analyses provide soil and rock pressure load demands with sufficient design load margins to address the modeling uncertainties.

5.1.1 FE Model of RB Structure

The structural FE model consisting of beam, shell, solid, and spring elements adequately represents the RB structural configuration for all main structural members. The FE model includes gross discontinuities such as large openings and member eccentricity. Thick shell elements are used to model the reinforced concrete shear walls, slabs and basemat. 3-D beam elements are used to model the reinforced concrete or steel columns, beams, and trusses. The shell and beam elements are established at the centerline of the wall, slab, beam, column, and truss elements. Rigid beam and shell elements or rigid links are used to model member eccentricities and offsets.

Linear elastic contact springs connect the RB structural and subgrade FE models. Stiffness properties are assigned to the contact springs to adequately represent the interaction mechanism between the structure, the water proofing material and the soil as described in Section 5.1.2.

Results obtained from these contact spring elements serve for calculation of soil pressures on the below grade RB shaft exterior wall. The results obtained from the contact spring elements serve to:

- validate the earth pressure loads considered by the design as described in Section 5.1.3, and
- determine whether separation between RB shaft wall and soils occurs in the static and dynamic loadings as discussed in Section 5.3.9.

The mesh of the FE models is sufficiently refined to produce stress demand calculations that are not significantly affected by a further refinement of the FE size or the shape. Finer meshes are used around penetrations and openings that are larger than half of the wall or slab thickness. Meshes of major walls and slabs consists of at least four shell elements along the short direction and at least six shell elements along the long direction.

The FE models used for seismic SSI analyses have a sufficiently refined mesh to be capable of transmitting the entire frequency range of interest for the seismic design of the RB SSCs. In accordance with the requirements of ASCE\SEI 4-16 (Reference 8.7), Section 5.3.4, the FE mesh shall be smaller than or equal to one-fifth of the smallest wavelength transmitted through the soil model, i.e. the maximum mesh size:

$$d_{max} \leq \frac{V_S}{5\, f_{cutoff}} \qquad (5\text{-}1)$$

where: V_S is the shear wave velocity of the transmitting soil material; and

f_{cutoff} is the cutoff frequency of analysis determined as described in Section 5.3.2

Larger element sizes may be used when justified as described in Section 5.3.4 of ASCE\SEI 4-16. Stiffness properties are assigned to structural members in the RB FE model in terms of Young's modulus and Poisson ratio that are determined in accordance with the governing design codes:

- American Concrete Institute ACI-349-13 (Reference 8.24) for the reinforced concrete members; and
- AISC N690-18 (Reference 8.25) for the steel and steel-plate composite (SC) members.

5.1.2 Soil-Structure Interaction Modeling Assumptions

Several simplified assumptions are introduced in the SSI design analyses of RB FE model to enable an efficient calculation of stress demands on the RB structure due to pressure loads from soil and rock surrounding and supporting the RB shaft. The following are the main assumptions for subgrade modeling used for the design SSI analyses performed following the SASSI methodology:

1) The properties of the subgrade materials are assumed to be isotropic and linear elastic;
2) The non-linearities at soil-structure interfaces are neglected;
3) The rock mass is assumed continuous and the presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones is neglected;

4) The static lateral pressures on the RB shaft due to the weight of self-supporting rock (i.e., excavated rock that does not require lateral support) can be neglected.

As described in Section 5.2.1, an approach is used for the development of linearized properties of soil and rock materials for the 1-g static SSI analysis to provide upper bound estimates of the demands on the RB structural members. Upper bound structural deformations and stress demands and lateral soil pressures on the RB below-grade exterior walls are estimated by using upper bound values for the soil unit weight and soil and rock Poisson's ratio paired with lower bound values of soil and rock elastic moduli.

The following stiffness properties are assigned to the contact springs at the SSI interfaces in the RB FE model for 1-g design analysis to provide upper bound lateral soil pressures on the RB below-grade exterior walls:

- The contact springs in the direction normal to the RB exterior walls are assigned properties representing upper bound stiffness conditions at the SSI interfaces; and
- The friction at the RB exterior walls is neglected by assigning very low stiffness properties to the contact springs in vertical and tangential direction.

The soil and rock strata in the SSI models used for calculating demands for design of RB structure are modeled based on the principles of continuum mechanics using isotropic linear elastic properties. Possible fracture zones, joints, bedding planes, discontinuities and cavities in the rock are not explicitly included in the design SSI analyses models. The stiffness properties assigned to the rock materials are developed, as described in Section 5.2.1.2, using empirical engineering and geomechanical rock mass classifications that quantitatively characterize the geologic and engineering parameters of rock masses.

The approaches described in Section 5.2.1.2 to calculate the equivalent linear properties of rock are applicable to structures that are relatively large compared to the block size of the rock mass and assumes the closely spaced discontinuities have similar characteristics where isotropic behavior of the rock mass is valid. When the discontinuity spacing is large compared to the dimensions of the excavation, the potential for unstable blocks or wedges and swelling or squeezing rock units need to be evaluated. The size of potentially unstable rock blocks and wedges should be estimated using an appropriate method (e.g., Reference 8.69). The evaluation of the potential loads from rock blocks and wedges may be completed using:

- the nonlinear FIA that includes rock/rock discontinuities represented by interface models described in Section 4.3.1.2; or
- static or pseudostatic force equilibrium analysis.

A simple example of a model for force equilibrium analysis of rock stability is provided in Section 5.1.4.3.

Strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness may be self-supporting even if some reinforcement is required to ensure a safe excavation. Typically, rock masses will yield slightly during construction – even with well-placed reinforcement – and arching will reduce the lateral loads except in highly fractured, weak, swelling, or squeezing rocks.

Because it is much more economical to reinforce the rock mass than to support it, rock reinforcement is used to create a self-supporting rock mass when the natural rock mass is not self-supporting. Reinforcement like tensioned and untensioned anchors may be installed inside the rock mass to help the rock mass support itself by eliminating progressive failure along planes of low strength as described in USACE 1110-1-2907 (Reference 8.26). Frequently, the reinforcement addresses specific rock wedges (keying) or is designed to form a beam or arch within the rock to create a stable, self-supporting excavation. Surface treatments such as shotcrete, strapping, and mesh may also be used for stabilization, protection of exposed rock, and control of loosened rock.

The design of the BWRX-300 considers this rock reinforcement as initial ground support that is separate from the permanent ground support system because the rock reinforcements and any surface protection may be inaccessible after construction. Therefore, the design addresses the rock loads remaining after the initial ground support degrades by including the potential weight of the solid rock in the design 1-g SSI analysis based on the results of non-linear FIA as described in Section 5.1.3.

Additional design analysis may be performed where earth pressure loads are applied to the below grade exterior walls of the refined RB structural model to account for:

- the effects on the RB design of anisotropic or heterogenous rock responses that cannot be directly modeled by the isotropic elastic models used for the one-step design SSI analysis; or
- potential pressures from unstable blocks of rock mass.

The magnitude and distribution of these additional earth pressure loads are determined from the results of the nonlinear FIA or force equilibrium analyses of the unstable rock mass. The structural design demands obtained from this additional earth pressure analysis are combined with the results of the one-step SSI analysis to ensure the RB structural design adequately addresses the effects of anisotropic and heterogenous rock behavior and accounts for potential unstable rock mass loads.

The SSI analysis of RB FE model are performed for a set of subgrade profiles to account for the variability and uncertainties in the subgrade material properties in accordance with the regulatory guidance of SRP 3.7.2 Subsection II.4 and ASCE/SEI 4-16 (Reference 8.7), Section 5.1.4. To address the effects of primary non-linearity, soil dynamic properties are used that are compatible to the free-field strains generated by a typical design level earthquake. These strain-compatible properties are developed as described in Section 5.2.4.

The effects of secondary non-linearity induced in the soil and rock by the structural vibration are neglected because in general, the structural vibration induces plastic deformations of the soil and dissipation of energy in the SSI system that reduces the structural response as shown in Reference 8.27 and Reference 8.28. On the other hand, the secondary non-linearity of subgrade materials may amplify the magnitude of the dynamic lateral pressures. The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones within the rock mass may also affect the stability of individual blocks or the rock mass during an earthquake that can potentially amplify the seismic rock pressure loads. Section 5.3.11 describes the approach used to evaluate the effects of subgrade materials non-linearity on the seismic response and design

- degradation of other soil support system.

This additional rock load on the RB shaft wall may be uniform with contact grouting to avoid stress concentration or point load associated with the block or wedge that is reinforced to stabilize the rock excavation. The evaluation of these rock pressure loads assumes that the excavation has reached stability with initial rock support and that the liner will accept 100 percent of the initial rock support as it relaxes over the lifetime of the structure. These loads should be conservative because rock loads in stressed rock masses are typically not following (e.g., they are not independent of displacement and typically reduce with displacement due to arching). The notable exception would be due to the presence of hydrostatic loads and swelling or squeezing rock displacements that may continue to apply a large load with continued displacement.

The presence of discontinuities may also affect the load transfer from adjacent shallow or surface founded structures to deeper structures. This potential load transfer is dependent on the geometry of the discontinuities, surface structure and embedded structure. When the additional load from the surface structure may be transferred to a potentially unstable rock block or wedge, this additional load should be included in the determination of reinforcement and the potential rock load on the exterior of the shaft or the rock block or wedge may be over-excavated and backfilled to reduce the load. Consideration of the geometry of the load transfer may allow the surface structures to be re-arranged to reduce or eliminate this load transfer to a potentially unstable rock block or wedge.

If cavities are present at the deployment site, sensitivity analysis are also performed by varying locations and sizes of cavities to address the effects of potential cavities on the rock pressure demands on the RB structure during operation.

The pressure load validation FIA uses the constitutive models described in Section 4.2 to represent the non-linear response of soil and rock subgrade materials, and the models described in Section 4.3.1 to represent the response at interfaces including the interfaces of RB structure with the surrounding subgrade. Because the intent of the FIA is to calculate best estimates of the soil and rock pressure loads, constitutive and interface models are developed using best estimate soil and rock properties obtained from the results of site investigation and laboratory testing programs described in Section 3.1. The stiffness of the RB structure in the FIA models is calculated per the governing design codes. Conservative design values obtained from the literature can also be used for certain input parameters.

A best estimate soil and rock pressure profile on the RB shaft is developed as an envelope of all maximum lateral pressure values calculated by the non-linear FIA of all analyzed post-construction stages and scenarios. This lateral pressure profile is compared to the lateral pressure profile developed from the results of the linear elastic 1-g design analysis to confirm the equivalent linear elastic model provides adequately conservative loads for the structural design. Soil and rock design pressure margins are calculated based upon the minimum values and the distribution of the ratio between the design soil and rock pressures obtained from the 1-g linear elastic analysis and the best estimate pressures obtained from the non-linear FIA. If the values of the calculated soil and rock design load margins are below the values deemed adequate to address the uncertainties and variations of subgrade properties, the rock mass weight or the equivalent linear soil and rock stiffness properties used for the 1-g design analysis are adjusted. Adequate values of the soil and rock design load margins are established based on the uncertainties and

variability of soil and rock properties used as input for the non-linear FIA and the significance of the non-linear and anisotropic response of subgrade materials on the soil and rock pressure demands.

If the results of non-linear static FIA indicate that the non-linear and anisotropic effects have a significant effect on the rock soil pressures and the site is characterized by a high seismicity, sensitivity SSI analyses are performed on non-linear models, as described in Section 5.3.11, to assess the effects of non-linear soil and rock response on the dynamic lateral pressure demands.

5.1.4 Probabilistic Earth Pressure Analyses

Probabilistic analyses may be performed to demonstrate that the magnitude of earth pressures used for the design are adequate to address uncertainties in the pressure load calculations. The external wall of the RB that is contact with soil is subdivided into discrete regions. The general approach consists on computing the probability density function of the subgrade pressure at each discrete region to calculate the probability distributions of soil and rock pressure loads on the RB below-grade exterior walls.

The probabilistic earth pressure load analysis addresses two types of uncertainties in the calculations of earth pressure loads:

- Parameter uncertainties related to natural randomness and uncertainties in measurements of mechanical properties of in-situ subgrade materials; and
- Model uncertainties related to the models used for earth pressure calculations.

Parameter uncertainty includes random variability of measured parameters including spatial variability and systematic measurement errors as well as uncertainties related to the methods used for the development of site subgrade parameters from empirical relationships. The random variability is manifested as the scatter of the data around a mean trend and is composed of the spatial variation of the subgrade properties and random measurement errors. Because the random measurement errors are often not distinguishable from spatial variation of the subgrade properties, they are usually considered jointly. Systematic error is divided into:

- Statistical error in the mean that can be reduced with increasing the sample size and number of measurements and tests being performed
- Bias in sampling and measurement procedures that is corrected by means of correction techniques/algorithms
- Bias introduced by the method used for development of subgrade parameters that is addressed by considering different approaches and empirical equations to calculate discrete probability distributions that are then combined as described in Subsection 5.1.4.4.

The model uncertainty that represents the uncertainty related to the model's ability to accurately predict the soil and rock pressures is manifested as a bias error in the earth pressure calculations. In general, the model uncreate is reduced by using more sophisticated models and an increasing number of model parameters. On the other hand, the increasing number of parameters used in the sophisticated models increases the parameter uncertainty and may reduce the overall confidence in the calculated soil pressure results. The model uncertainty is approached by means of

Table 5-1: Models for Probabilistic Earth Pressure Analyses

Subgrade Type	Site Parameter (x_i)	Model
soil	unit weight	Analytical equations
	cohesion	
	friction angle	
rock	rock mass properties	Force equilibrium, FE or a finite difference model
	unit weight	
	cohesion	
	friction angle	
	weak zone orientation	
	weak zone area	

Simple models that do not require explicit calculations of the state of strain and stress in the ground materials, are used for the probabilistic analyses of earth pressures on the RB shaft in contact with subgrade materials which mechanical properties are assumed to be continuous. For example, the following three models can be used to calculate lateral earth pressure coefficients representing three possible states:

a. at-rest condition representing essentially no movement of the structure relative to the surrounding subgrade;
b. active condition when the structure moves away from the surrounding subgrade; and
c. passive condition when the structure moves towards the surrounding subgrade.

These simple models provide probabilistic earth pressure distributions from the probabilistic distributions of the basic subgrade material strength parameters, the internal friction angle (φ), the cohesion (c) and the friction angle (φ_w) between the subgrade and RB cavity wall.

Force equilibrium models are used for probabilistic analysis of rock masses with discontinuities that may control the stability of individual blocks or the rock mass when the orientation is disadvantageous. Depending on the geometry of the discontinuities relative to the free face of the excavation, one or more blocks may slide along the discontinuities.

As shown on Figure 5-1, the sliding of the rock block driven by the surcharge load and its own weight is resisted by:

- the resistance force along the rock discontinuity due to cohesion (c_d) and the friction represented by the friction angle (φ_d); and
- the resultant of pressure loads at the rock-structure interface.

6.1 Control Building, Turbine Building and Radwaste Building Design Bases

CB, TB and RwB structures and foundations are designed in accordance with their seismic classification:

- Non-Seismic Category for the CB and TB structures; and
- RW-IIa Category for the RwB structure.

6.1.1 Non-Seismic Control Building and Turbine Building Structures and Foundations Design Bases

The non-seismic CB and TB structures are designed in accordance with the IBC (Reference 8.53). IBC (Reference 8.53), Chapter 16 provides structural design requirements, including those related to structural loads and load combinations, which also rely on applicable provisions from ASCE 7-16 (Reference 8.54). IBC (Reference 8.53), Section 1901.2 states that structural concrete shall be designed in accordance with the requirements of IBC Chapter 19 and ACI 318-14 (Reference 8.55) as amended in Section 1905 of the IBC. Concrete structures include the reinforced concrete foundations of the CB and TB, TB concrete pedestal on an independent foundation, and TB concrete walls used for radiation shielding and missile protection. The control room may also consist of a reinforced concrete structure within the steel framed structure of the CB. Section 2205.1 of IBC (Reference 8.53) invokes AISC 360 (Reference 8.58) for the design, fabrication and erection of structural steel elements in buildings, structures and portions thereof. Steel structures include the CB and TB steel braced frames with roof deck diaphragms.

In accordance with IBC (Reference 8.53), Table 1604.5, both the CB and the non-seismic portion of the TB are designated Risk Category IV structures. The CB and TB are considered part of a power-generating station required as emergency backup for Risk Category IV structures. Furthermore, the control room in the CB is designated as an emergency shelter to protect the inhabitants of the control room both during and after an earthquake, tornado, or hurricane.

IBC (Reference 8.53), Section 1609, describes requirements for determining wind loads and invokes ASCE 7-16 (Reference 8.54), Chapters 26 to 30. It should be noted that ASCE 7-16, Section 26.14, states that tornadoes have not been considered in its wind load provisions. Commentary in ASCE 7-16, Section C26.14, provides a discussion of tornado wind loads for building owners that may desire providing a greater level of occupant protection or minimizing building damage caused by tornadoes. Commentary in ASCE 7-16, Section C26.14.2, discusses the differences in wind pressures induced by tornadoes versus other windstorms.

The IBC (Reference 8.53) does not require the consideration of tornado wind loads except for structures designated as storm shelters, whose requirements are described in IBC (Reference 8.53), Section 423. IBC Section 1604.10, states that loads and load combinations on storm shelters shall be determined in accordance with ICC 500 (Reference 8.56). ICC 500 (Reference 8.56), Figure 304.2(1), provides tornado shelter design wind speeds, which range from 130 mph to 250 mph. ICC 500, Figure 304.2(2) provides hurricane shelter design wind speeds, which range from 160 mph to 235 mph. Of particular importance when designing for tornado wind loading is the effect of atmospheric pressure change (APC), which is described in ICC 500, Section 304.7.

IBC, Section 1613, describes requirements for determining earthquake loads and invokes ASCE 7-16 (Reference 8.54), Chapters 11, 12, 13, 15, 17 and 18, as applicable. The seismic

design category of a structure may be determined according with IBC, Section 1613, or ASCE 7-16.

6.1.2 Radwaste Category IIa Building Structure and Foundations Design Basis

The RwB is classified as a RW-IIa structure because it contains SSCs used for managing and containment of highly radioactive gas, liquid, and solid materials whose failure, considering the maximum inventory, would result in a potential unmitigated radiological release levels that may be higher than those specified in RG 1.143, Section 5.1.

In accordance with RG 1.143, Table 1 guidance, the design of the BWRX-300 RwB steel structures follows the provisions of AISC N690 (Reference 8.25). The design of the RwB concrete structures and basemat is in accordance with ACI 349-13 (Reference 8.24). Based on RG 1.143, Table 2, the loads for the design RW-IIa RwB structure includes:

- one-half of the SSE seismic load;
- wind load according to ASCE 7-16 (Reference 8.54)* for a Risk Category III structure;

  (*Note: ASCE 7-95 is referenced in RG 1.143, but most recently, ASCE 7-16 is used)
- tornado wind load equal to three-fifths of load provided in RG 1.76, Table 1; and
- Schedule 40 pipe and automobile tornado missiles based on SRP 3.5.1.4, "Missiles Generated by Tornadoes and Extreme Winds," Revision 4.

RG 1.143, Table 2, does not specify specific hurricane wind loading beyond that found using ASCE 7-16 (Reference 8.54) for a Risk Category III structure. RG 1.143, Table 3, provides the design load combinations based on the safety class of RW-IIa. RG 1.143, Table 4, provides guidance for calculating the design capacity based on safety class and the governing code/standard.

6.2 II/I Seismic Interaction Evaluations

The II/I seismic interaction evaluations of CB, TB and RwB structures are performed to ensure:

- the integrity of structural members of CB, TB and RwB lateral load resisting system under SSE loading is not compromised;
- the stability of CB, TB and RwB foundations under SSE loading is not compromised; and
- the gap distances between the CB, TB and RwB with the RB are adequate to prevent physical interactions between the buildings.

The II/I seismic interaction evaluations are based on seismic responses of CB, TB and RwB obtained from SASSI analyses of linear elastic FE models, which are refined sufficiently to provide accurate stress demands on the major lateral load resisting structural members and accurate seismic displacements in the direction of the adjacent RB. These SASSI analyses are performed on surface mounted models that neglect the effect of basemat embedment using PBSRS defined ground motions time histories and strain-compatible soil properties that are developed as described in Sections 5.2.3 and 5.2.4, respectively.

In lieu of SSI analyses, fixed base analyses can be performed, if the site-specific conditions meet any of the following criteria in ASCE\SEI 4-16 (Reference 8.7), Section 5.1.1:

occupants, and the TB structure does not collapse and impair safety functions of the main steam line.

As described in Section 6.1.2, the design-basis tornado wind load for the RwB is equal to three-fifths of load provided in RG 1.76, Table 1, and the design basis standard is ACI 349-13 (Reference 8.24). ACI 349.3R (Reference 8.18) requires structures to remain elastic under analyzed loading, so there are no provisions to consider inelastic responses for non-seismic loading; therefore, the extreme wind II/I checks for the RwB are performed to determine the maximum deflections of the RwB due to the controlling extreme wind loading, while requiring the RwB to maintain a linear elastic response to the loading.

The SC-I RB is also designed for design-basis hurricane- and tornado-generated missiles, as applicable. The hurricane-generated missiles are based on the missile spectrum in RG 1.221, Table 1 and the corresponding missile velocities in RG 1.221, Table 2. The tornado-generated missiles are based on the missile spectrum and missile velocities in RG 1.76, Table 2. The missile spectrum used for hurricane- and tornado-generated missiles is the same, except the hurricane missile spectrum only considers the larger automobile missile and not the smaller automobile missile used for Region III tornadoes. Extreme wind missiles are not considered in II/I interaction checks for the following reasons:

- II/I interaction evaluations for extreme wind ensure no gross failure of CB, RwB, or TB structures, while missile loads would only result in localized effects;
- The missile spectrum considered in the design of RB would envelope the effects of any missiles generated by localized failure of CB or TB components/cladding;
- The RwB is designed for design-basis tornado-generated missiles; and
- Any safety-related SSCs housed in non-SC-I structures have adequately designed missile barriers.

6.4 Summary of Design Approach for II/I Interaction

The following aspects of the BWRX-300 graded design approach for II/I interaction of non-SC-I CB, TB and RwB with adjacent SC-I RB presented in this section of the report may be referenced during future licensing activities that are beyond the current guidelines in RG 1.29, "Seismic Design Classification," Revision 5:

(1) General criteria for design of Non-Seismic CB and TB structures provided in Section 6.1.1, including the requirements for determining seismic and wind design loads.

(2) General criteria for design of the RW-IIa RwB structure provided in Section 6.1.2, including the requirements for determining seismic, wind, tornado wind and missile design loads.

(3) Approach for seismic II/I interaction evaluations of CB, TB and RwB structures presented in Section 6.2, including criteria and recommendations for calculations of seismic stress demands and displacements.

7.3 Generic Profiles of Dynamic Subgrade Properties

Eight generic profiles define the following dynamic properties of subgrade materials as a function of depth for the BWRX-300 generic seismic design:

A. Total unit weight (w) representing best estimates of the combined weight of the saturated soil and the pore water

B. Strain-compatible shear wave velocities (V_S) shown on Figure 7-2

C. Poisson's ratio (ν) representative of saturated soil conditions shown on Figure 7-3

D. Compression wave velocities (V_P) shown on Figure 7-4

E. Strain-compatible damping shown on Figure 7-5

Stiffness and damping properties are compatible to the strains generated by a design level earthquake event. These strain-compatible properties are obtained from the results of the set of probabilistic site response analyses performed on the randomized base-case profiles of measured or small-strain V_S that were used to calculate the GDRS as described in Section 7.2.

The top softer soil layers in the SSI analysis subgrade profiles reflect saturated soil properties with values of ν ranging from 0.48 to 0.49. The maximum value of ν is kept below 0.49 to ensure the numerical stability of the SSI analysis results. The V_P profiles shown on Figure 7-4 are calculated from the following elastic theory equation based on the strain-compatible V_S and ν values:

$$V_P = V_S \sqrt{\frac{2(1-\nu)}{1-2\nu}} \tag{7-1}$$

The V_P profiles are representative of saturated soil conditions with the V_P of softer soil layers close to the value of water $V_P \approx 1600$ m/sec.

The generic profiles are categorized in terms of average measured small-strain shear wave velocity of the top 30 meters of soil ($\bar{V}s_{30}$) and the depths to the geological base-rock. For example, the generic profile 180-600 represents a generic site subgrade condition where the average measured (small-strain) shear wave velocity of top 30 m of soil $\bar{V}s_{30} = 180\ m/sec$ and the geological base-rock is located at depth approximately 600 m below the profile surface.

The eight generic profiles represent a range of generic site conditions varying from deep soft soil represented by Profile 180-600 to hard rock represented by Profile 2032-30 and cover a wide range of subgrade properties at the majority (>80%) of candidate sites.

A wide variation of shallow soil stiffness conditions is captured by considering profiles with:

- $\bar{V}s_{30} = 180\ m/sec$ representative of medium stiff soil sites
- $\bar{V}s_{30} = 270\ m/sec$ representative of firm soil sites
- $\bar{V}s_{30}$ = $400\ m/sec$ representative of stiff soil sites
- $\bar{V}s_{30}$ = $500\ m/sec$ and $760\ m/sec$ representative of soft rock sites
- $\bar{V}s_{30} = 900\ m/sec$ representative of firm rock sites

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