ENCLOSURE 1

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BWRX-300 Advanced Civil Construction and Design Approach

Non-Proprietary Information

GE Hitachi Nuclear Energy

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Licensing Topical Report

BWRX-300 Advanced Civil Construction and Design Approach

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REVISION SUMMARY

Revision Number	Description of Change
0	Initial Revision

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, and RG 1.138 “Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants,” Revision 3, 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.

   - 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.

B. 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.

C. 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.

D. 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.

- 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.

E. 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.

F. 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.

G. 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.

   - 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.

H. 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.

I. 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.

- 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.

- 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.

- 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.

- 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.

- 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), turbine-generator support systems and bridge crane.

The RwB, which houses the systems for management of 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.

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 do 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.

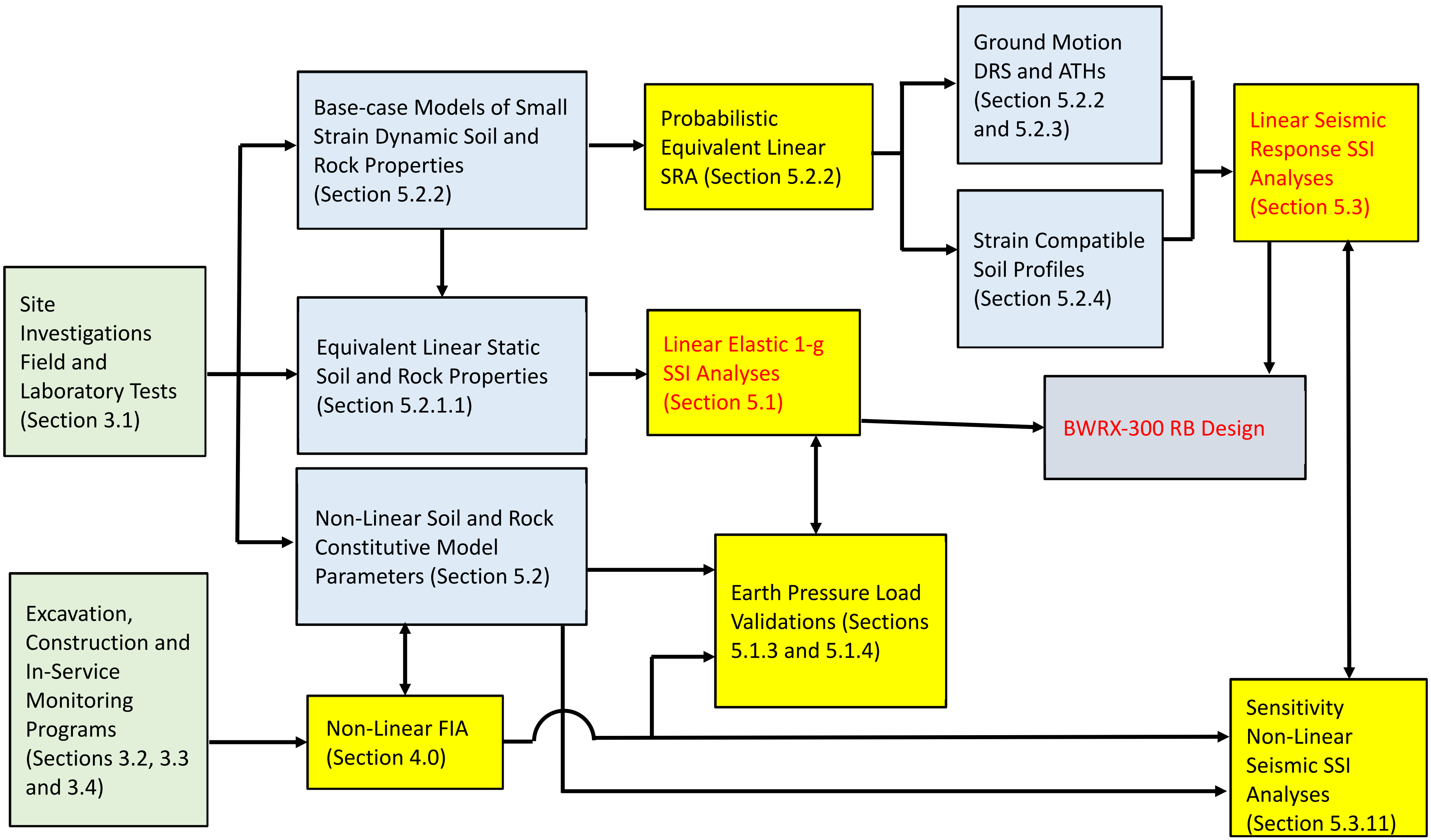


Figure 1-1: BWRX-300 Investigation, Monitoring, Analysis and Design Process

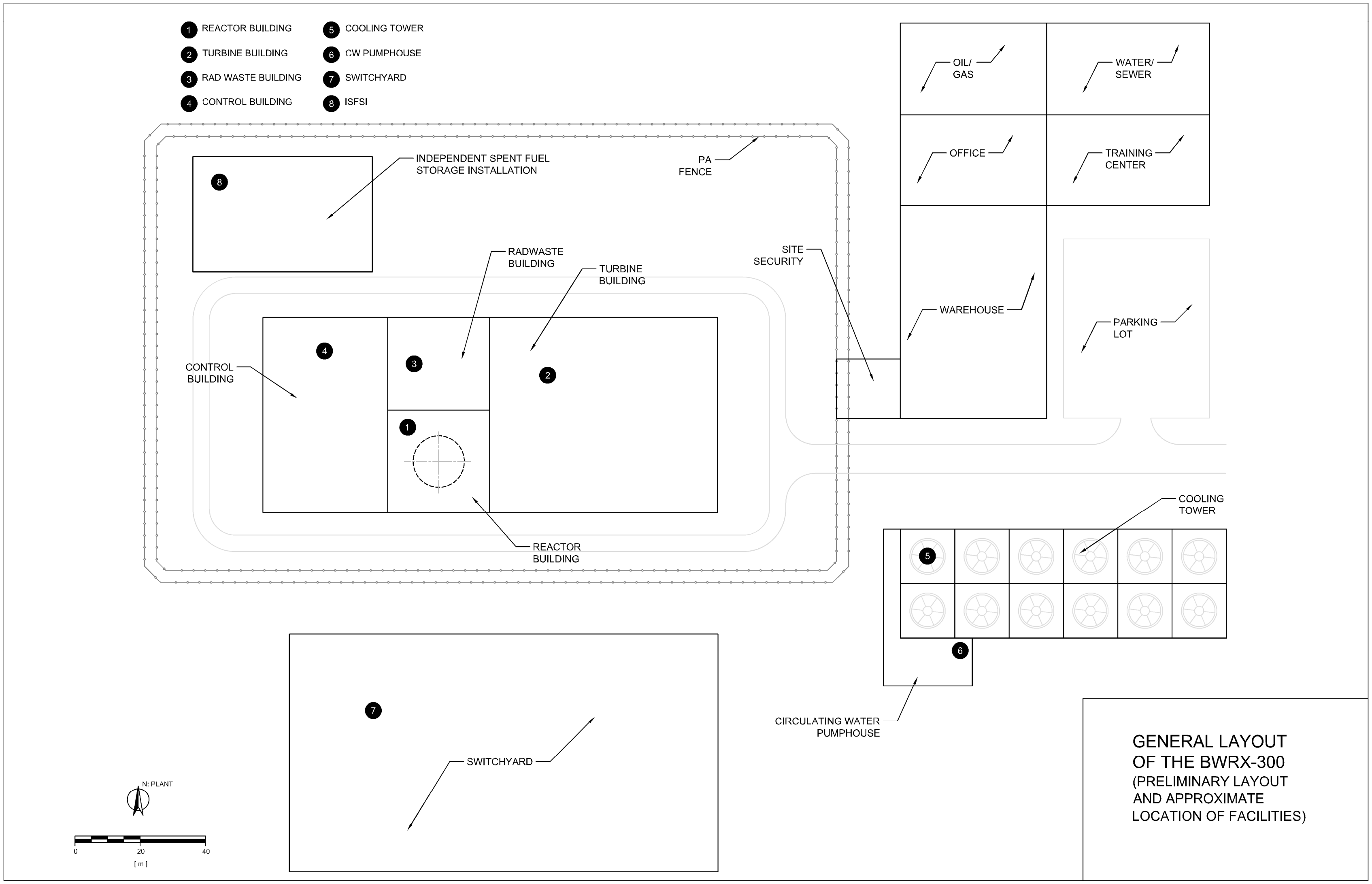


Figure 1-2: BWRX-300 Conceptual Site Plan

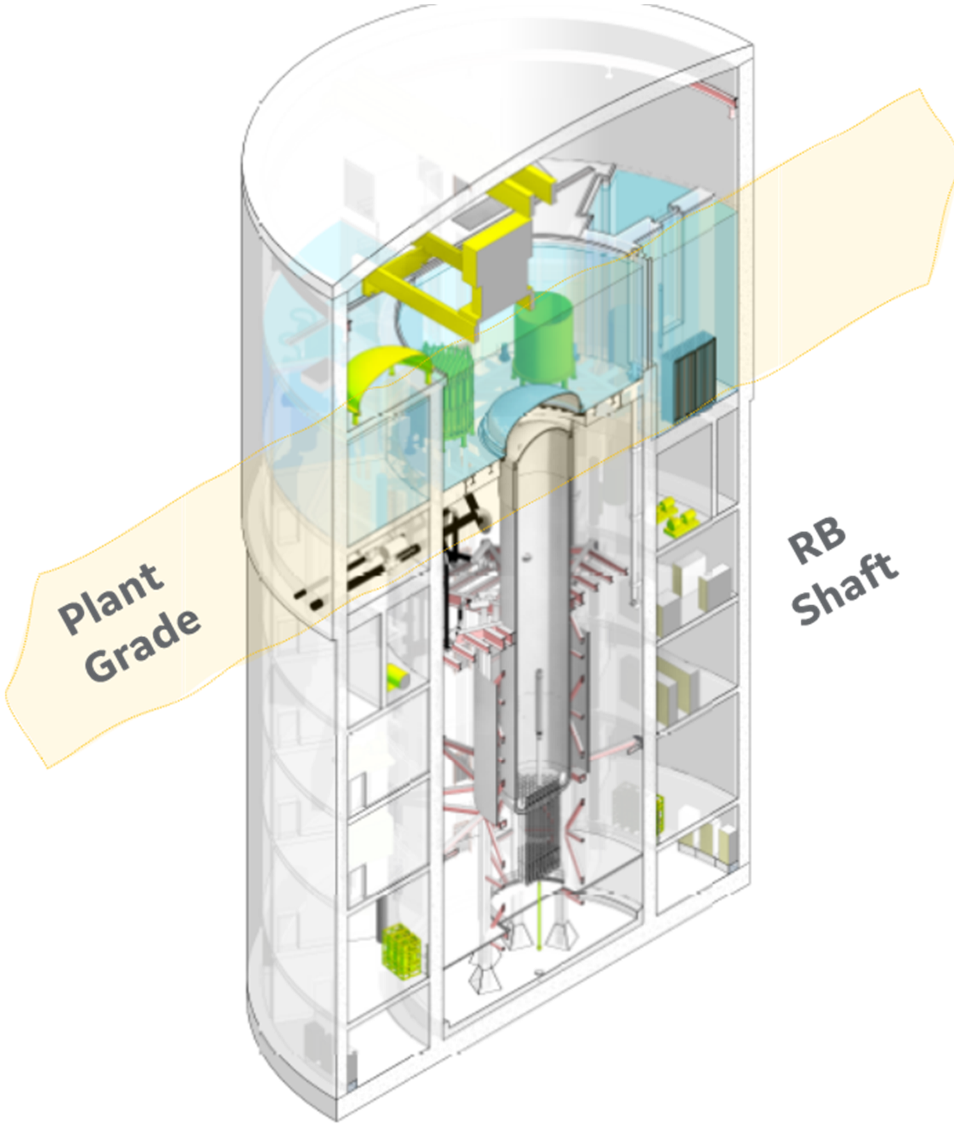


Figure 1-3: BWRX-300 Conceptual RB Three-Dimensional Section View

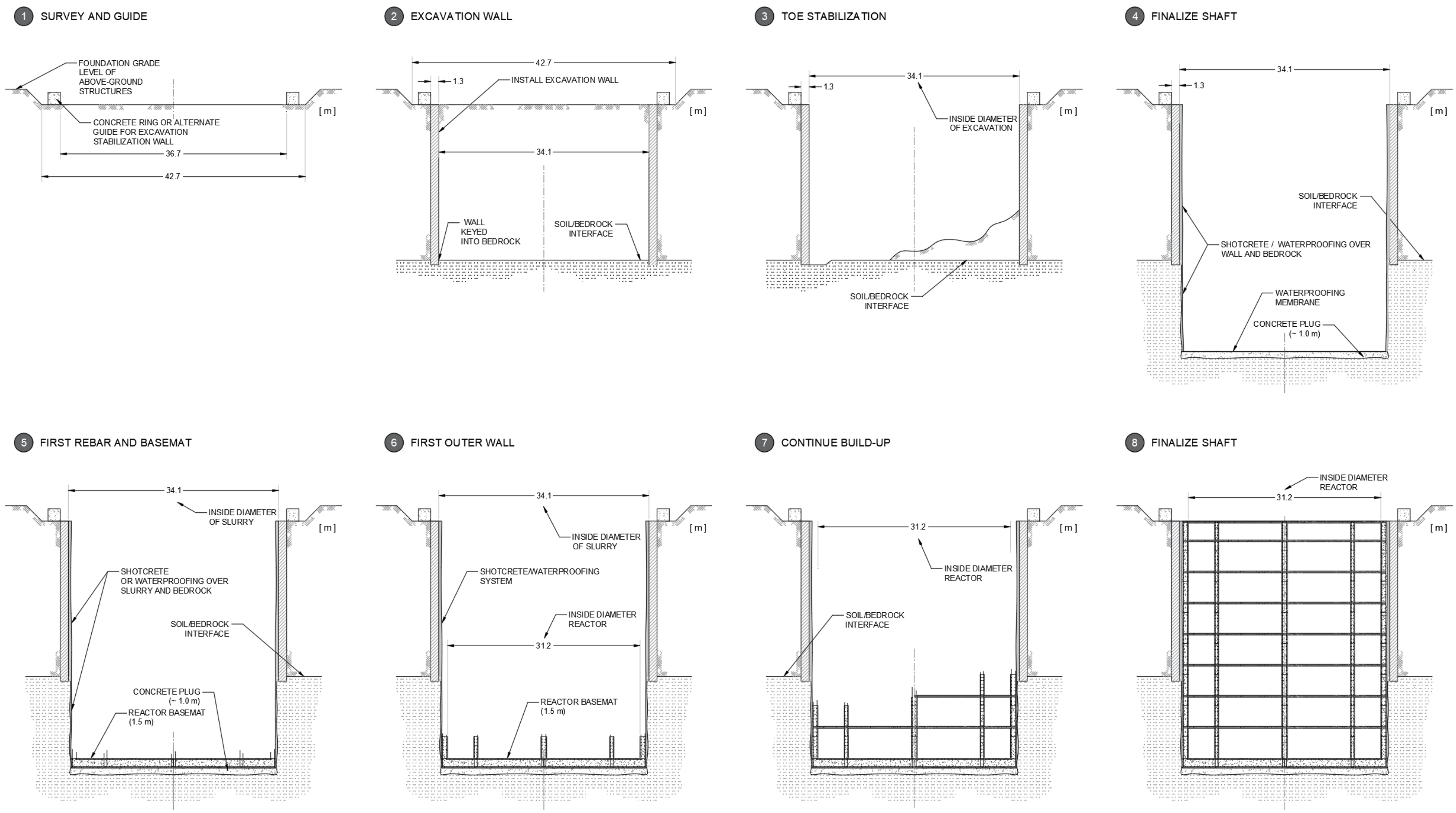


Figure 1-4: BWRX-300 Reactor Building Shaft Construction

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, 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 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 TB structure does not collapse to result in impairment of safety functions of the main steam piping or the OGS.

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 and RG 1.138.

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 and RG 1.138 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 FEA 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 and RG 1.138. 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). 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 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 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 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 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.

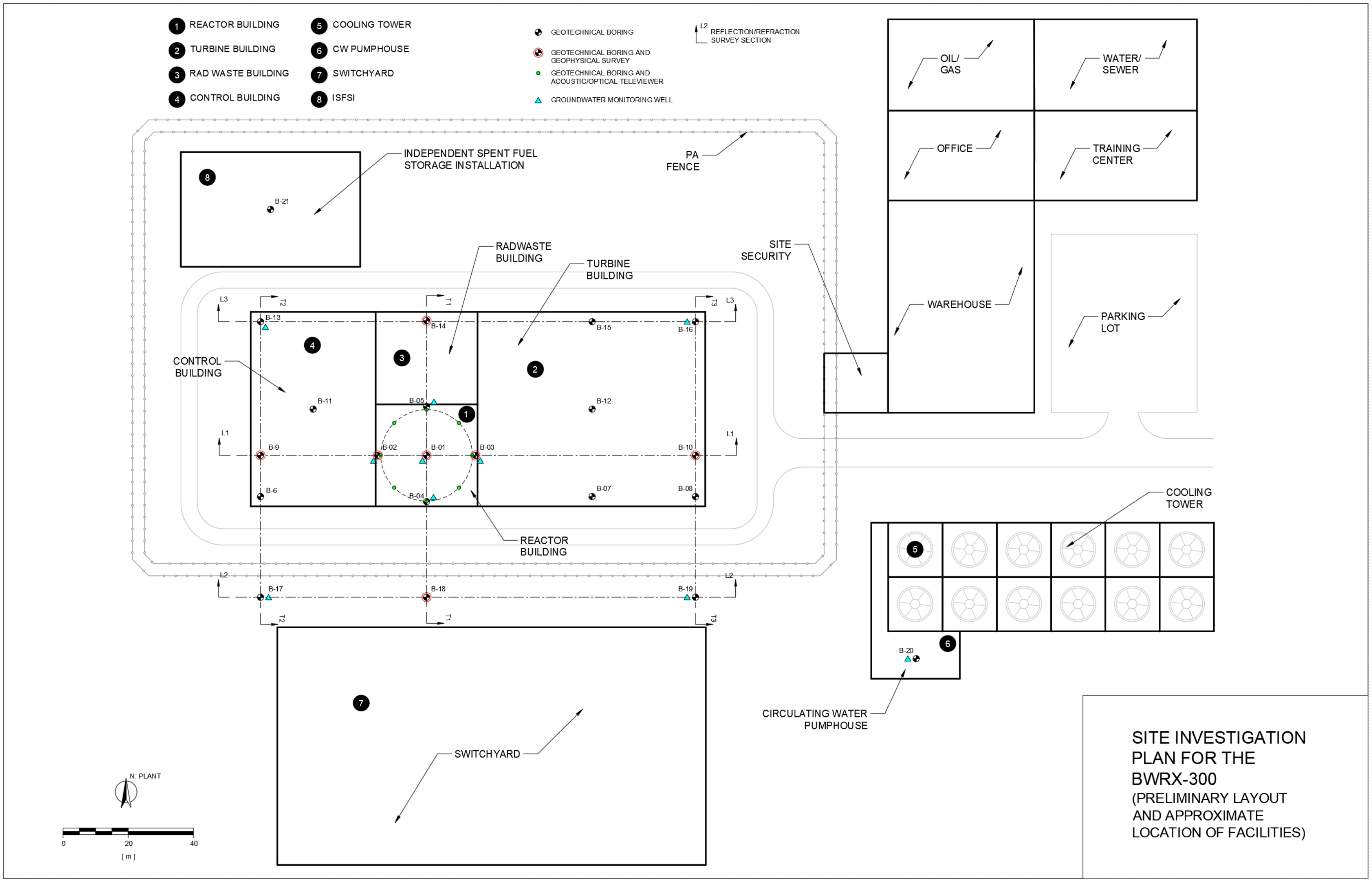


Figure 3-1: Preliminary Layout of the BWRX-300 Borings

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) − Perform pressuremeter tests on weak to moderately soft rock portions to have data parameter for estimation of elastic moduli − Measure in site 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 site 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 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.

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,
- 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;
- 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 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.

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 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, orientations, aperture, and other characteristics of the discontinuities. 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. 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 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:

- 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.
- 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).

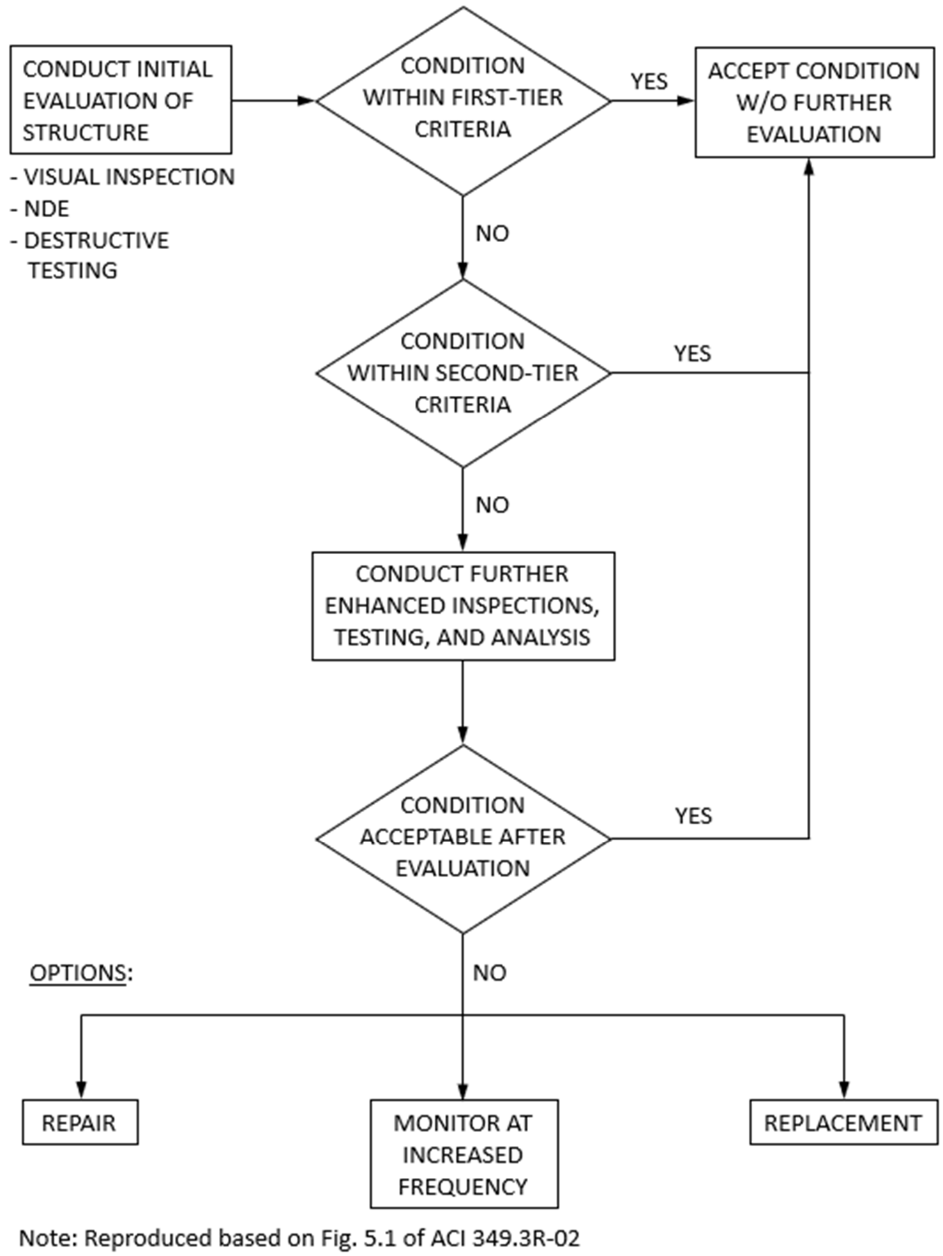


Figure 3-2: Framework of Structures Monitoring and Aging Management Program

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 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.

- 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.
- 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.

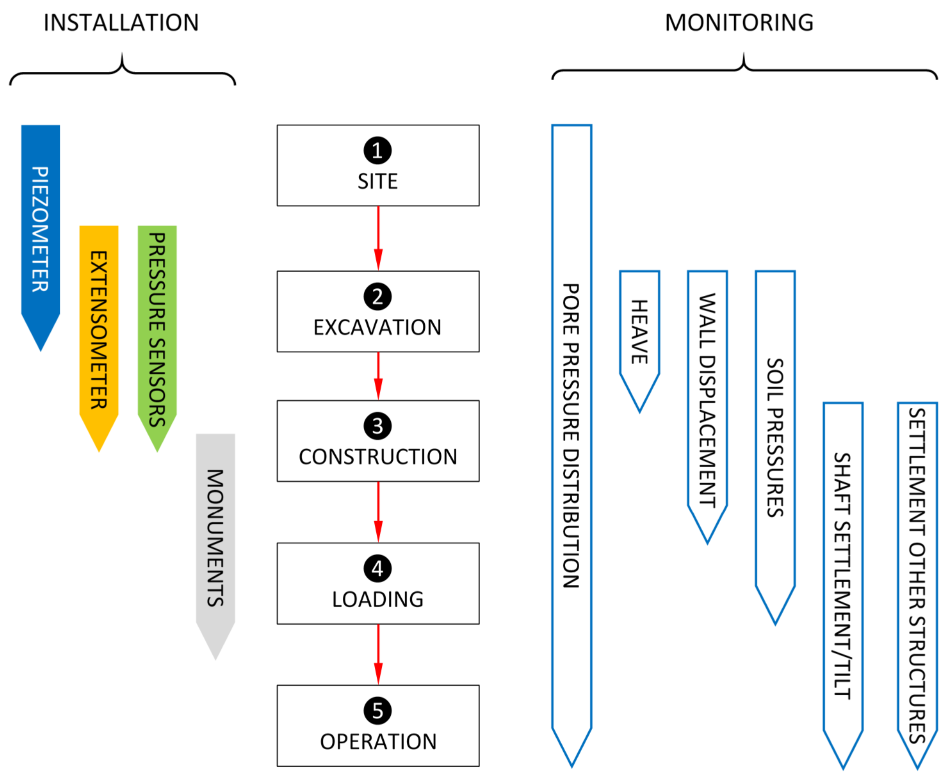


Figure 3-3: Field Instrumentation and Monitoring

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.

(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.

(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:

- Three dimensional
- 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.

- 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.

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:

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

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:

- 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. 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. Sensitivity evaluations may be conducted assuming lower friction and variations of the interface stiffness. These types of analyses provide insight to understand the uncertainty introduced by interfaces in the stress distribution and deformation response of the structure.

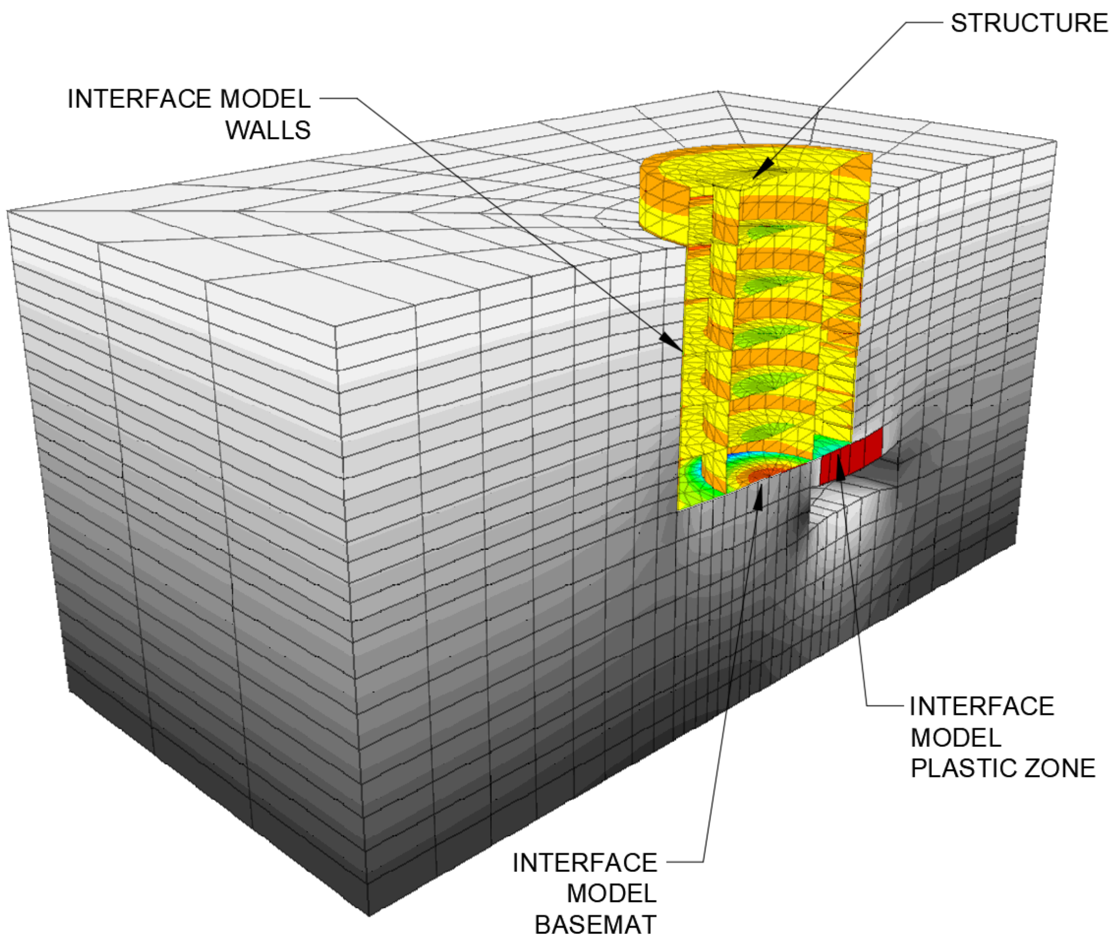
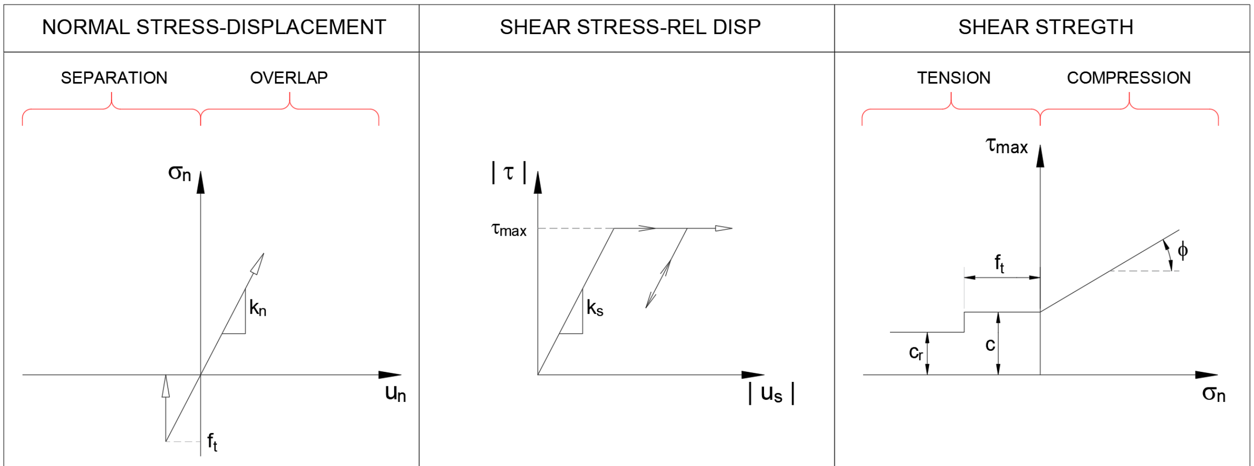


Figure 4-1: Location of Interfaces between Soil and Structure



σ_n - Normal stress
k_n - Normal coupling stiffness
u_n - Normal displacement
f_t - Normal coupling tensile strength
τ - Shear stress
u_s - Shear displacement
τ_{max} - Shear stress at plastic onset
k_s - Shear coupling stiffness
c - Shear coupling cohesion
c_r - Shear coupling residual cohesion
ϕ - Shear coupling friction angle

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. The parameters representing the slipping may also be estimated based on the properties of the weakest interface material. Strength reduction factor may be used to adjust the spring stiffness based on the roughness and residual strength of the interface when the sliding occurs.

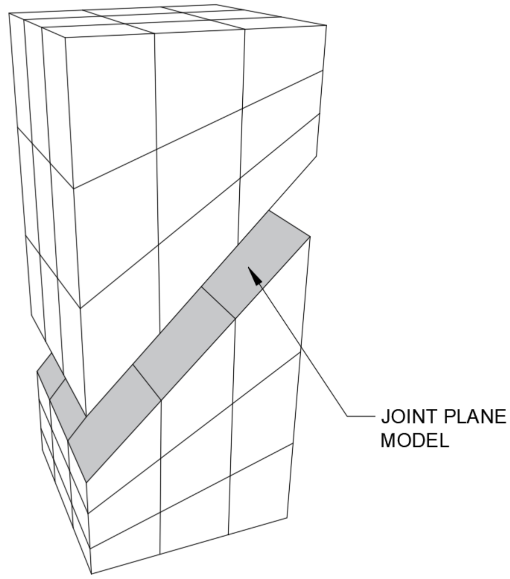


Figure 4-3: Joint Plane Model

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.

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.

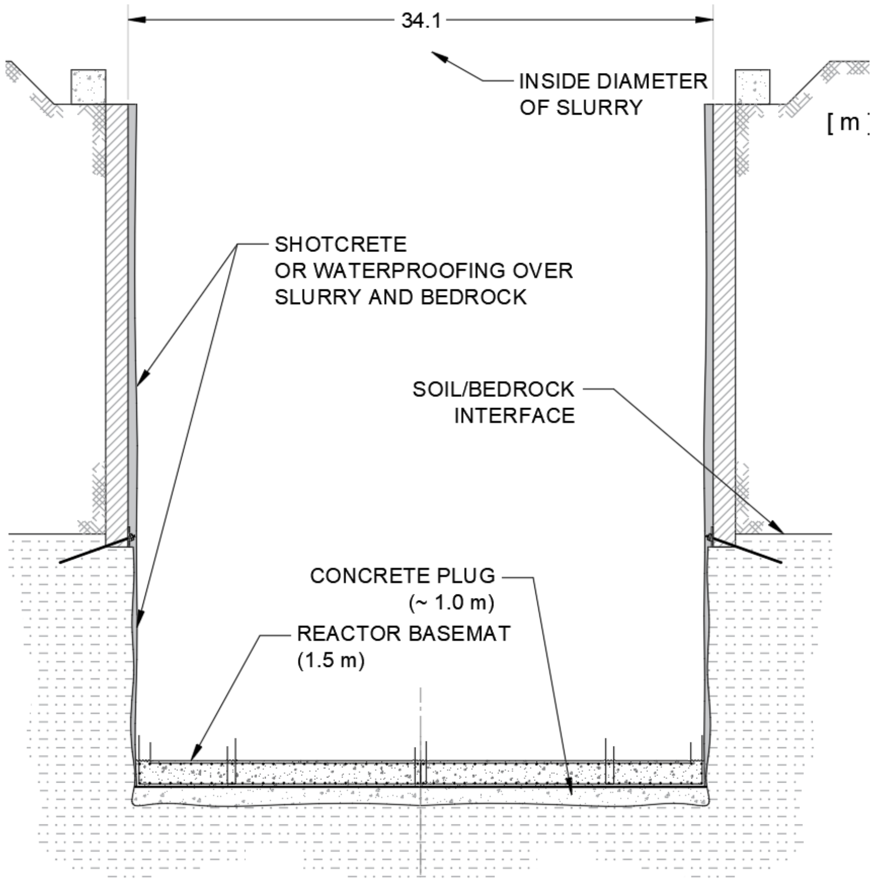


Figure 4-4: Excavation

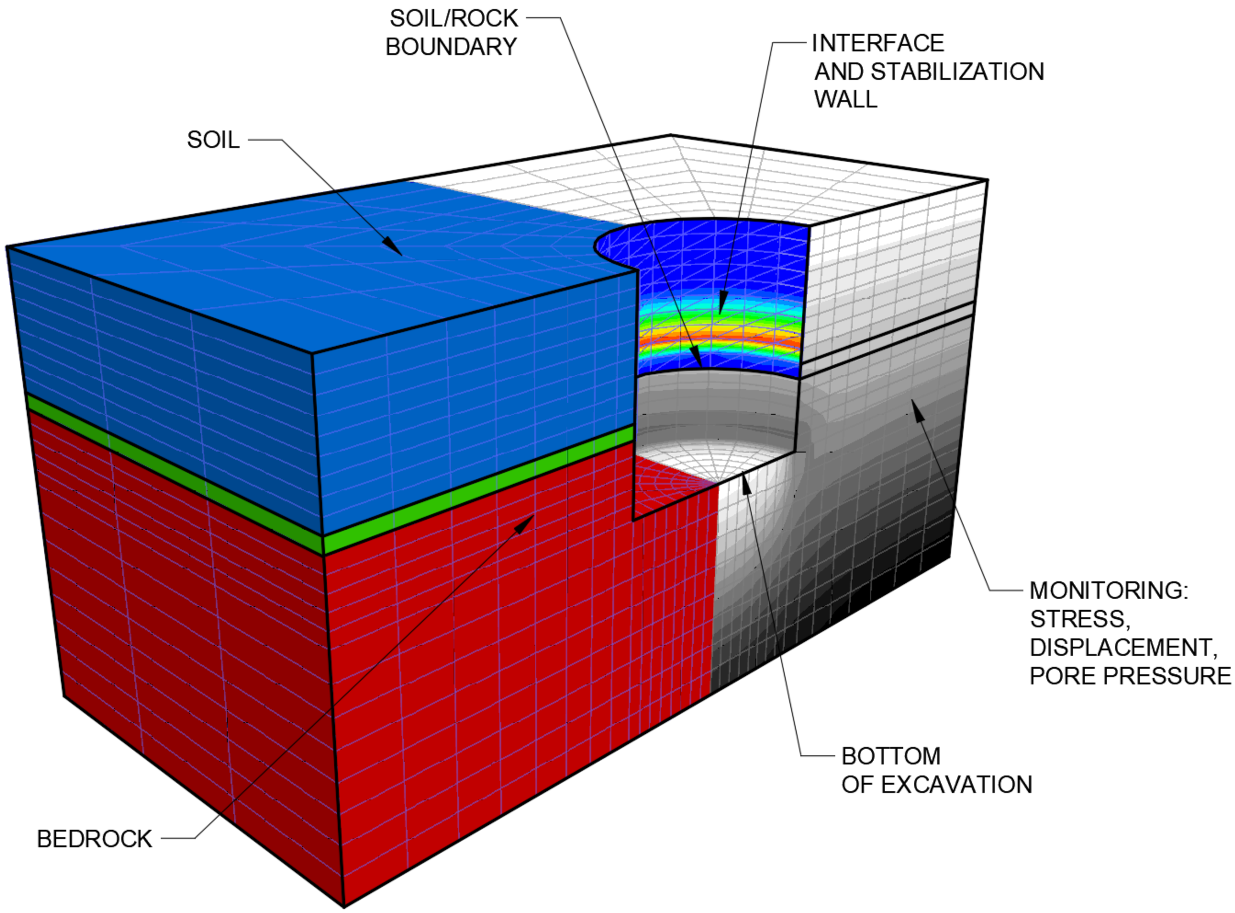


Figure 4-5: Excavation Modeling

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.

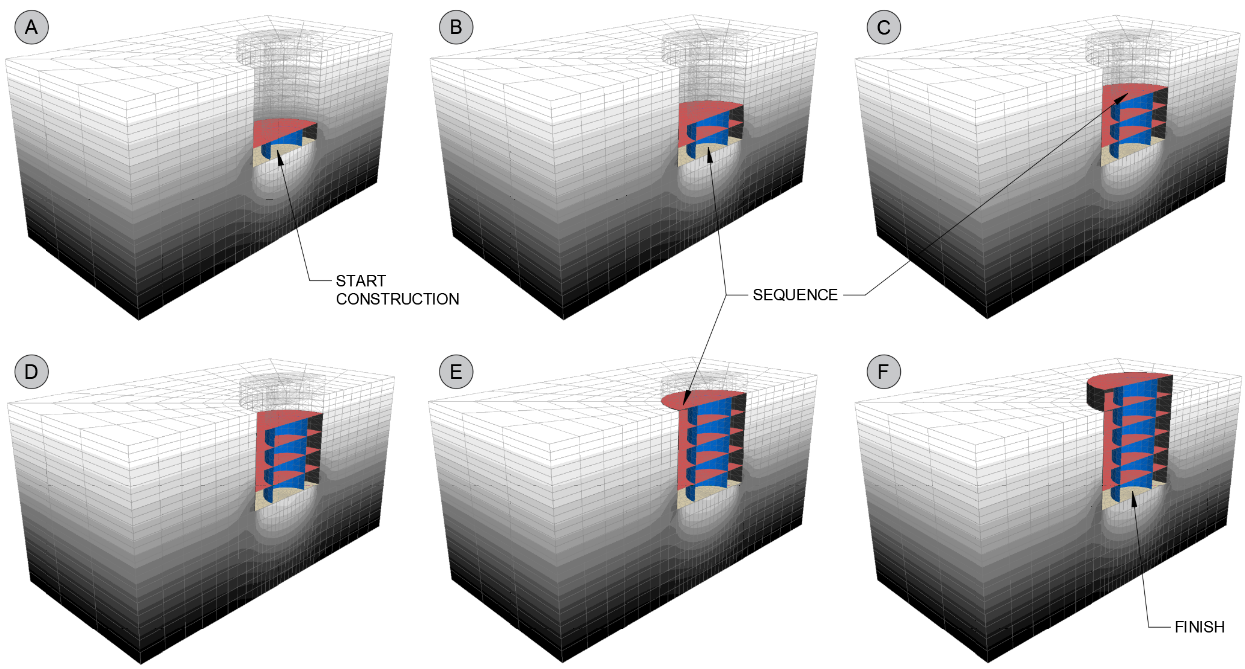


Figure 4-6: Modeling During Construction

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.

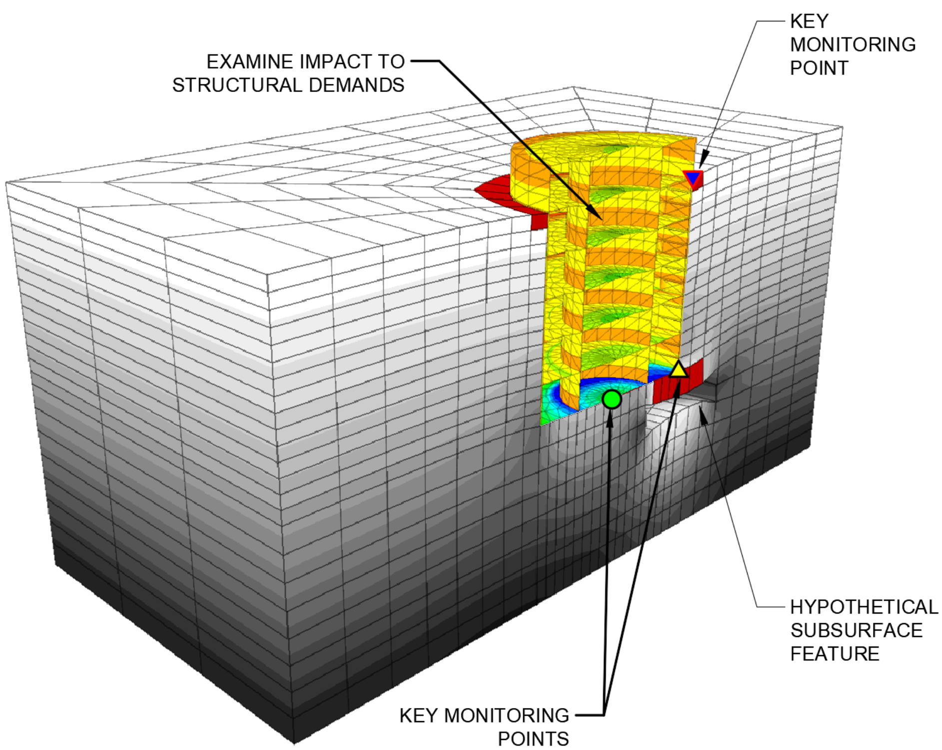


Figure 4-7: Modeling During Operation

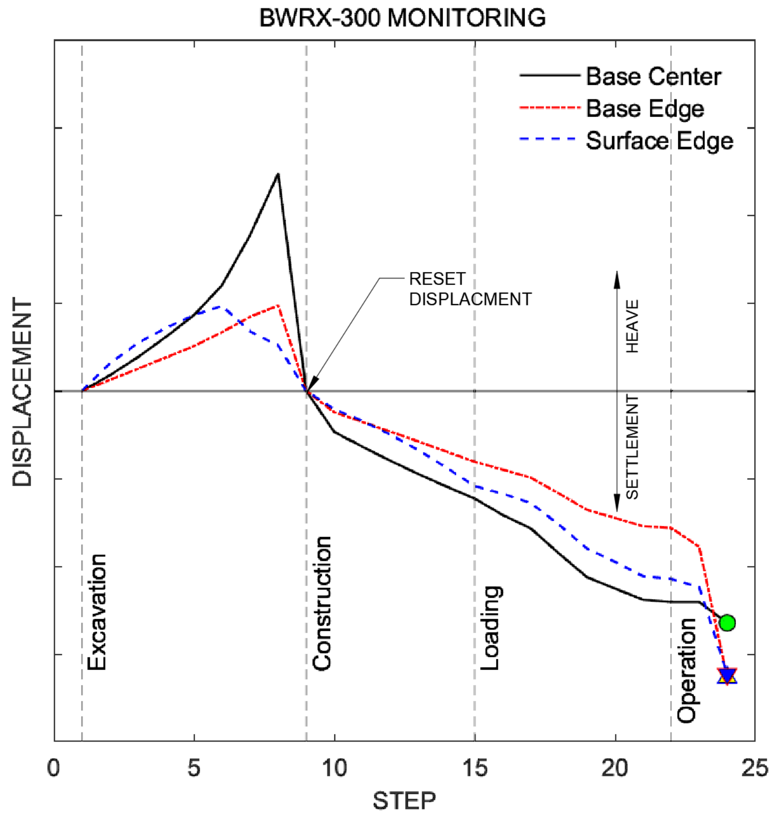


Figure 4-8: Hypothetical Results of Modeling During Operation

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. 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:

$$d_{max} \leq \frac{V_S}{5\, f_{cutoff}} \tag{5-1}$$

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 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 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 welling or squeezing rock units need to be evaluated. The evaluation of the potential loads from rock blocks and wedges may be completed using 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 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.

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. 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 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. 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

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 ($\bar{x}_1, \bar{x}_2 \dots \bar{x}_n$):

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

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:

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

where: x_{ik} is the k^th 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):

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

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:

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

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

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

where: x_{ik} and x_{jk} are the k[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)

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

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:

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

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:

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

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	unit weight	Force equilibrium, FE or a finite difference model
	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.

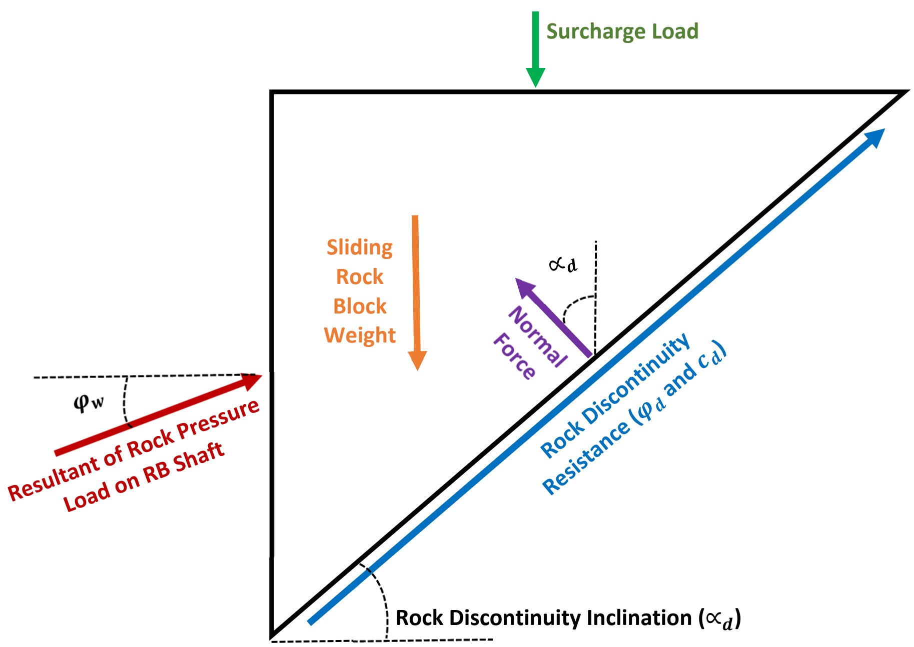


Figure 5-1: Force Equilibrium Model for Rock Wedge Analysis

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 ($R_{BP} = 0$ 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:

$$\upsilon_{st} = \frac{K_0}{1 + K_0} \tag{5-14}$$

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.

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 $N \le 15$ $E_{st} = 600\ (N + 6) + 2000$ for $N > 15$	N/A
Clayey Sand[1]	$E_{st} = 320\ (N + 15)$	$E_{st} = (3\ to\ 6)\ q_c$

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
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
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 (≤ 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:

$$\emptyset_{cr} = \frac{\pi}{2} - \arctan\sqrt{\frac{1}{1-2\,\upsilon}} \tag{5-26}$$

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:

$$p_{LB}(z) = 0.9\left[\frac{K_{0_{LB}}(z)}{K_0(z)} p_{1g}(z)\right] \qquad \text{for } z \le z_{gw}$$

$$p_{LB}(z) = \gamma_w\left(z - z_{gw}\right) + 0.9\left[\frac{K_{0_{LB}}(z)}{K_0(z)} p_{1g}(z)\right] \qquad \text{for } z > z_{gw} \qquad (5\text{-}29)$$

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 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.

6.1.1 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

RwB and the portion of the TB enclosing the main steam piping and OGS are classified as RW-IIa structures because they contain 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 Rw-IIa RwB and TB structures include:

- 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:

$$\Delta_{allow_i} = (\Delta_{gap} - \Delta_{tol}) - \Delta_{RB_i} \tag{6-1}$$

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:

$$\Delta_{II_i} = C_{NL}\,\Delta_{St_i} + \Delta_{Fe} + \Delta_{Fs_i} \tag{6-2}$$

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 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.

(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)

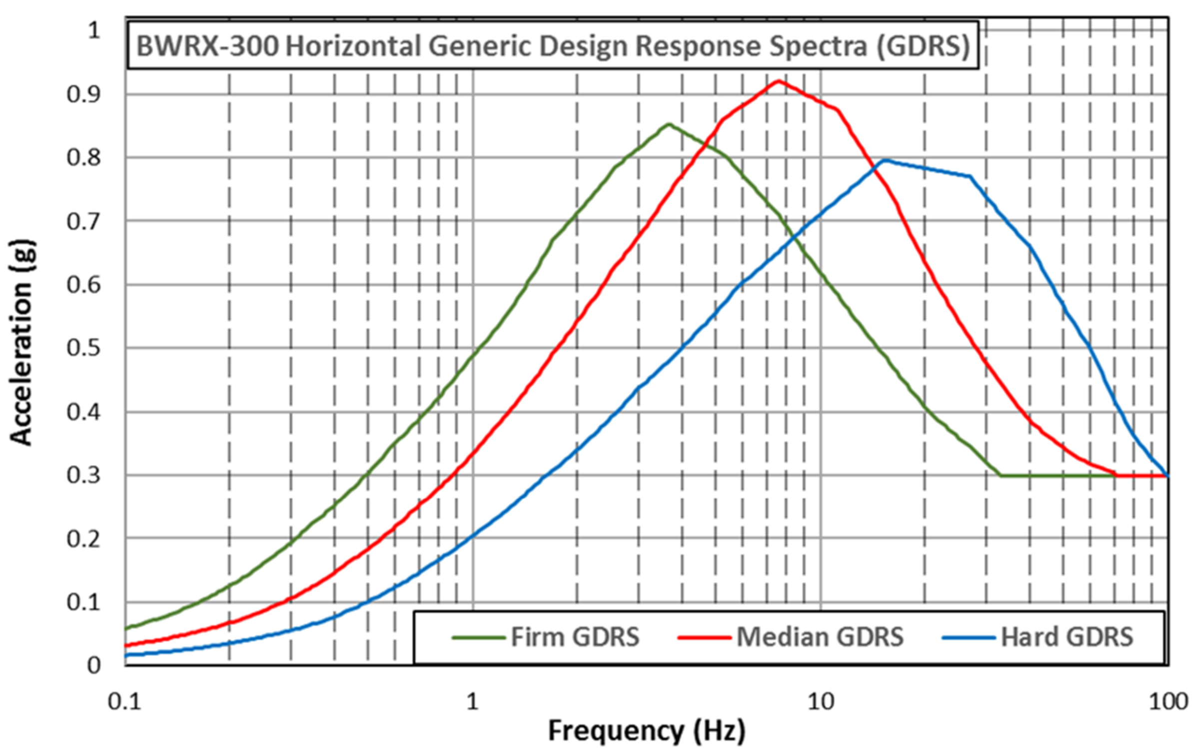
(4) WUS – Western United States, CEUS - Central and Eastern United States

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.



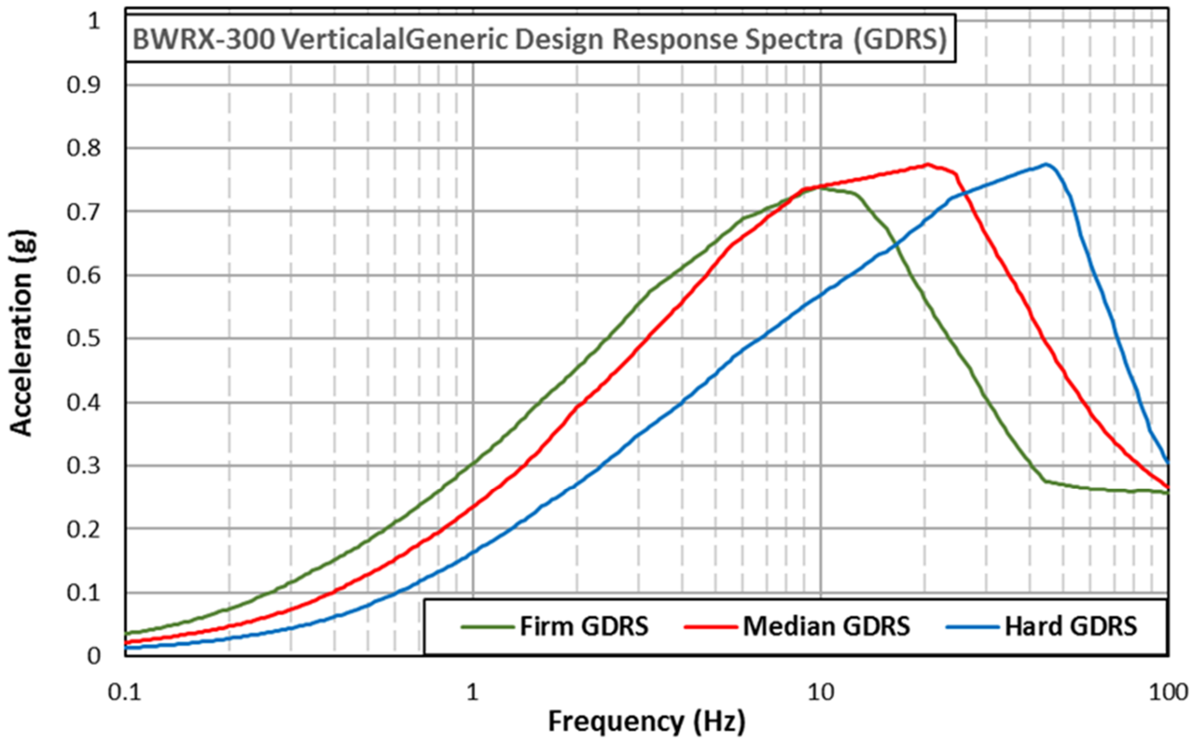


Figure 7-1: BWRX-300 Generic Design Response Spectra

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:

C. Total unit weight (w) representing best estimates of the combined weight of the saturated soil and the pore water

D. Strain-compatible shear wave velocities (V_S) shown on Figure 7-2

E. Poisson's ratio (ν) representative of saturated soil conditions shown on Figure 7-3

F. Compression wave velocities (V_P) shown on Figure 7-4

G. 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:

$$V_P = V_S \sqrt{\frac{2(1-\nu)}{1-2\nu}} \qquad (7\text{-}1)$$

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:

$$V_{S\_ave} = {H_S}\Big/{\sum_i {d_i}/{V_{Si}}} \tag{7-2}$$

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:

$$f_S = \frac{V_{S\_ave}}{4\, H_S} \tag{7-3}$$

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.

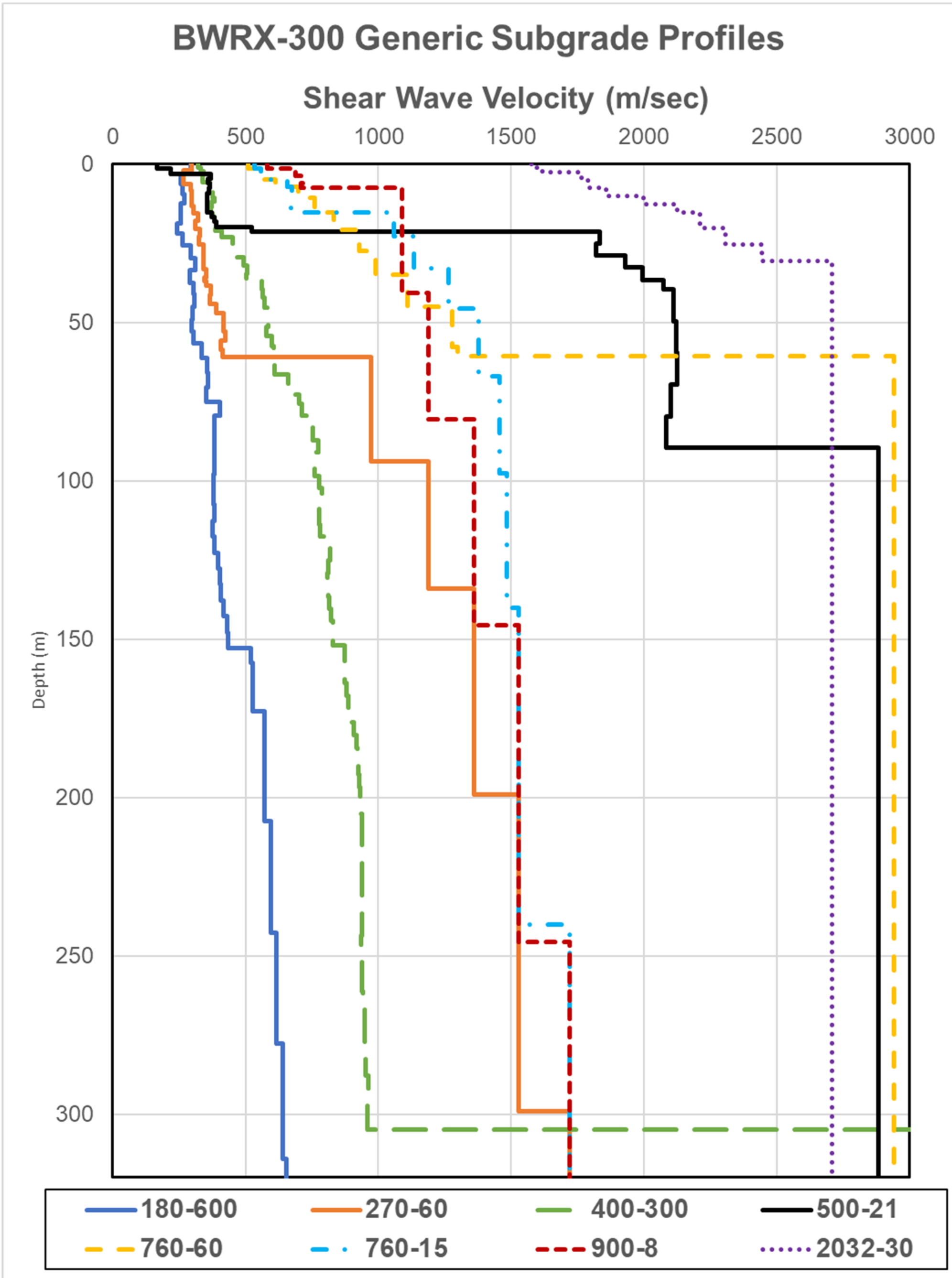


Figure 7-2: BWRX-300 Generic Shear Velocity Profiles

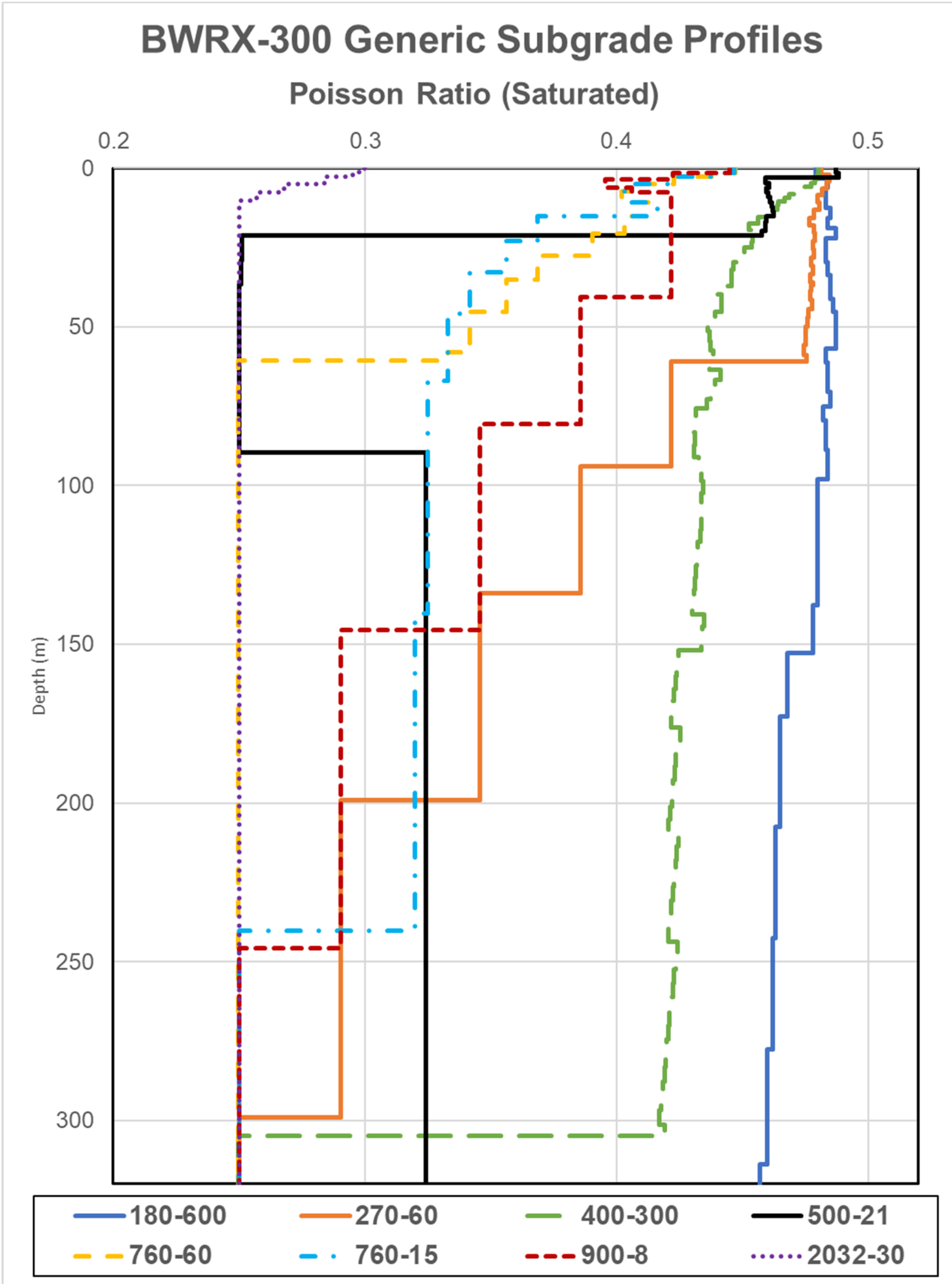


Figure 7-3: BWRX-300 Generic Saturated Soil Poisson Ratio Profiles

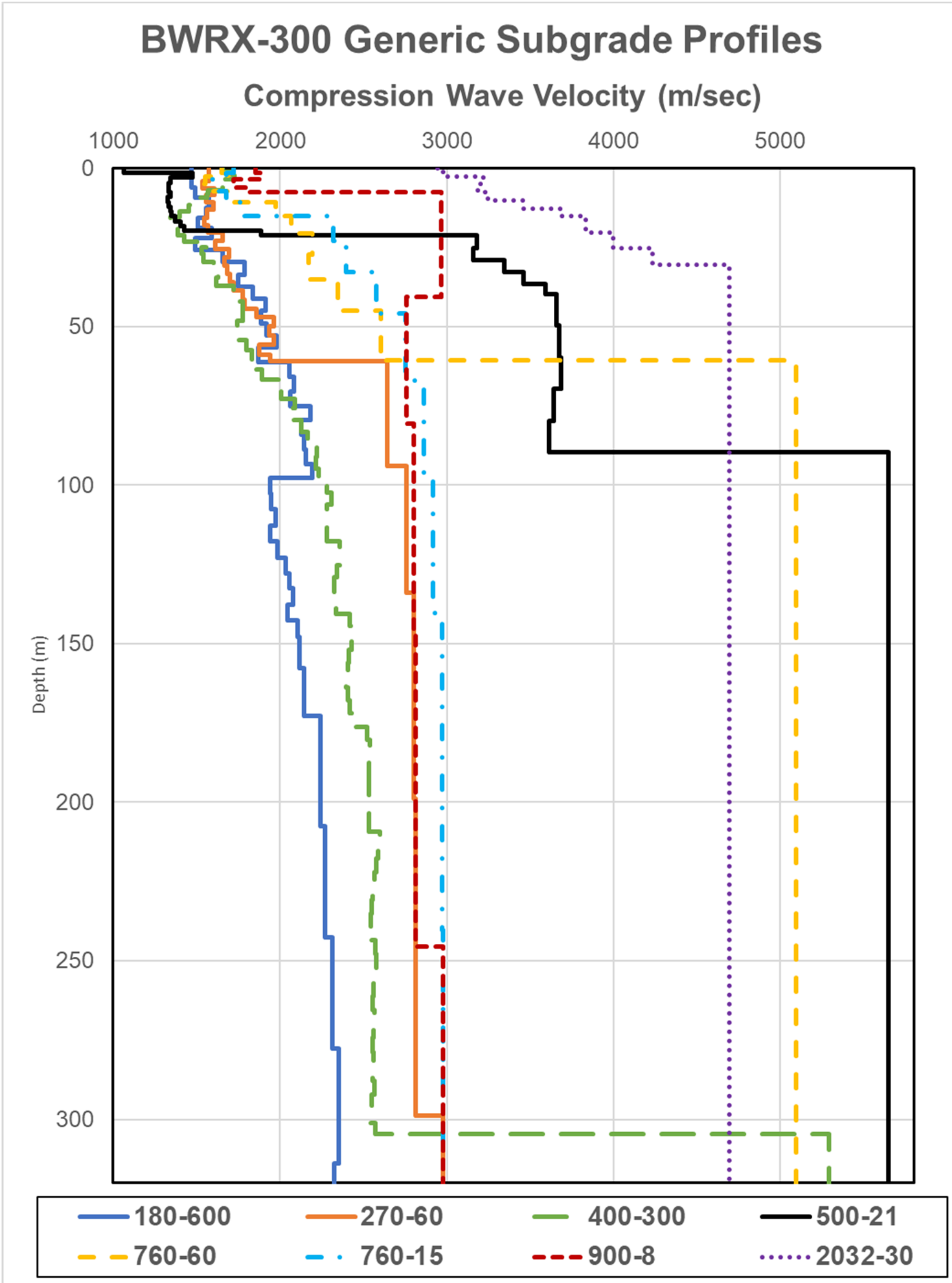


Figure 7-4: BWRX-300 Generic Compression Velocity Profiles

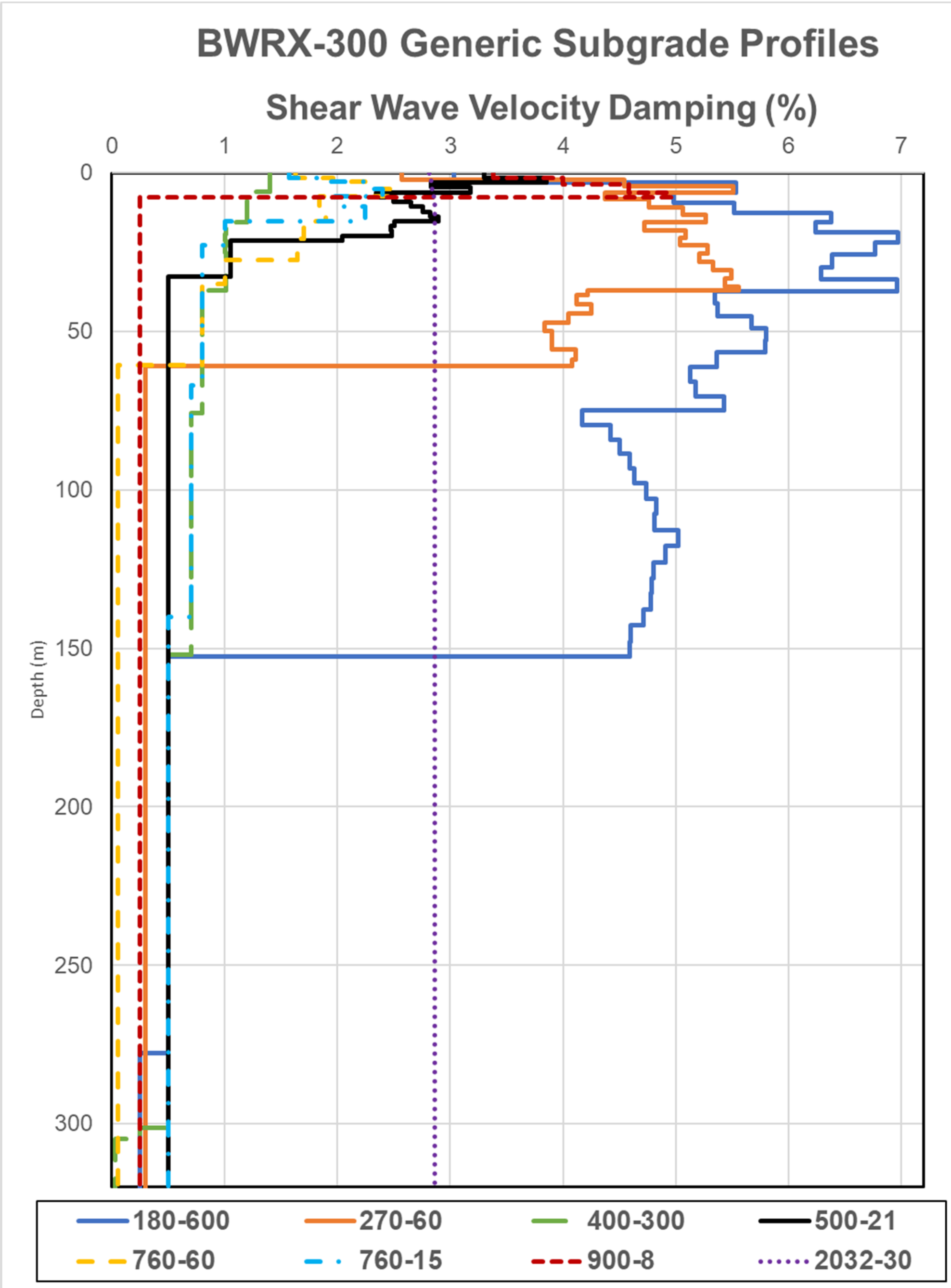


Figure 7-5: BWRX-300 Generic Damping Profiles

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.

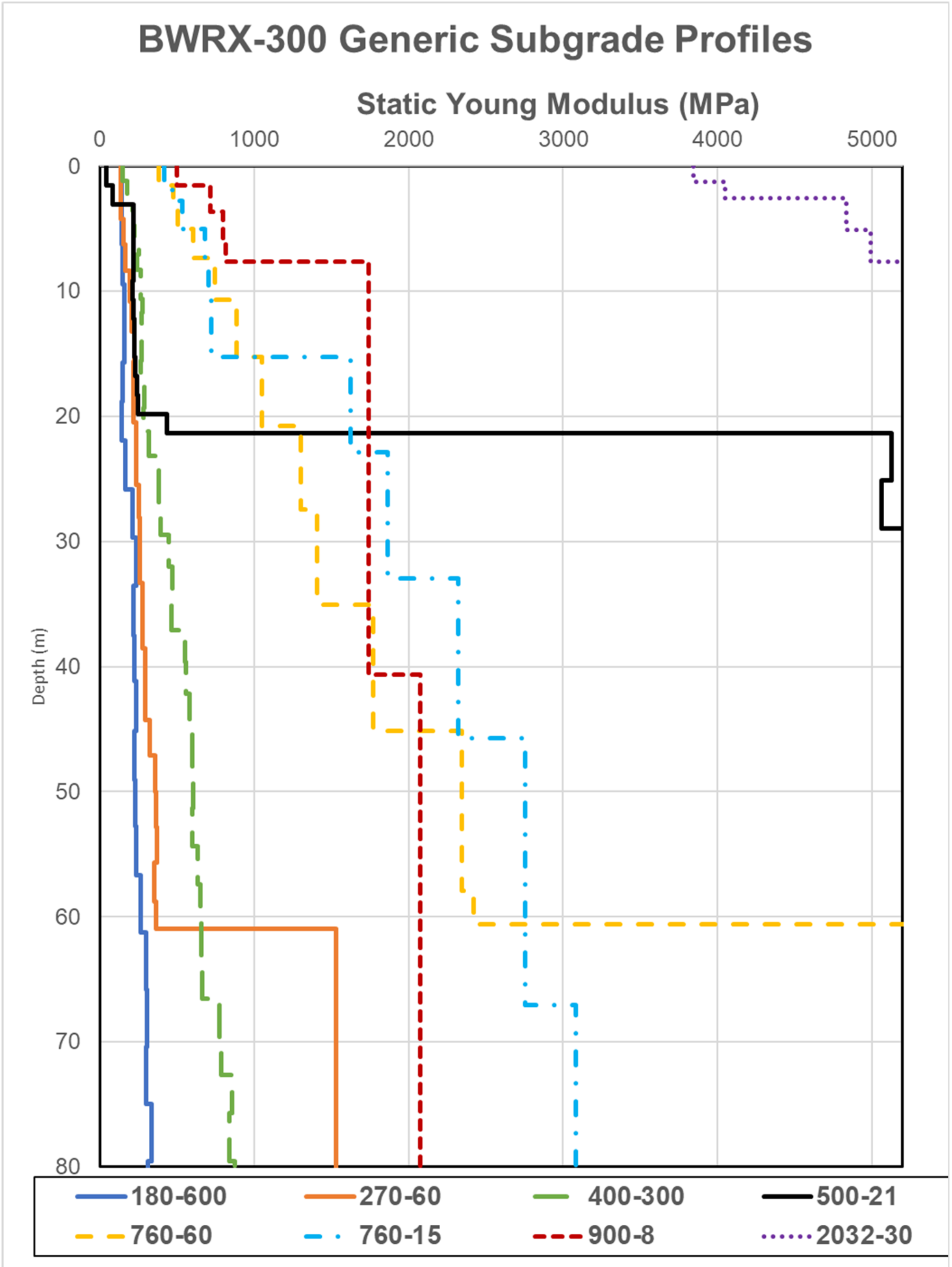


Figure 7-6: BWRX-300 Generic Dry Unit Weight Static Analysis Profiles

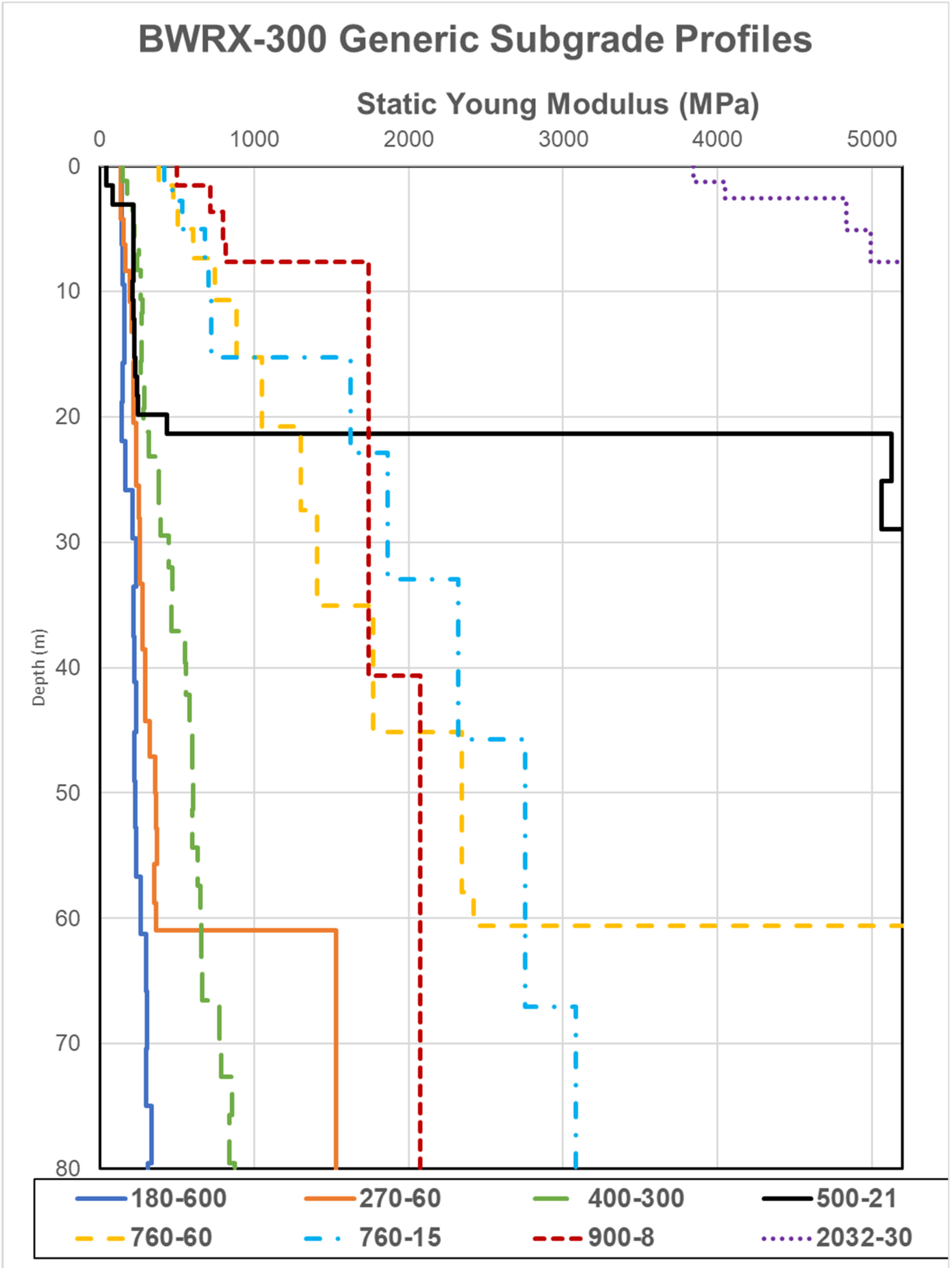


Figure 7-7: BWRX-300 Generic Young's Modulus Static Analysis Profiles

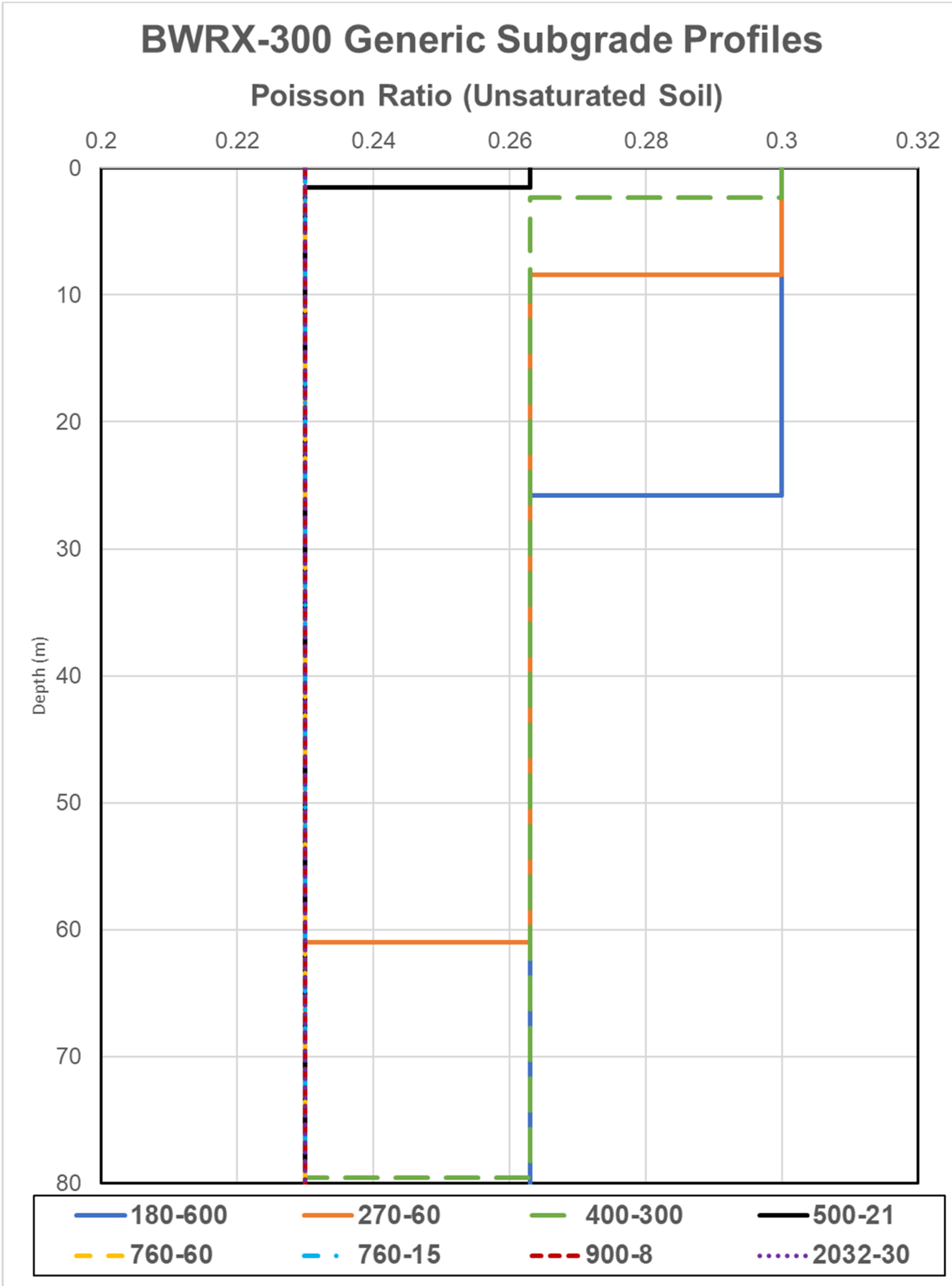


Figure 7-8: BWRX-300 Generic Dry Soil Poisson Ratio Static Analysis Profiles

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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