ENCLOSURE 2

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BWRX-300 Advanced Civil Construction and Design Approach

Non-Proprietary Information

information gaps between the existing available information and specific needs for the siting of the BWRX-300.

3.1.1 Site Investigation Program

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

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

In order to address the specific requirements of the BWRX-300 RB design, the subsurface site investigations are performed following the guidelines of RG 1.132 (Reference 8.64) for SC-I type site investigations considering the combined footprint areas of the RB SC-I foundation and the adjacent TB, CB and RwB foundations. The extended area considered by the BWRX-300 subsurface site investigation ensures an adequate characterization of the subsurface conditions under the TB, RwB and CB foundations and resulting surcharge loads, which are important for the design of the deeply embedded RB structure and seismic design of RB SC-I SSCs.

Appendix D of RG 1.132 (Reference 8.64), Spacing and Depth of Subsurface Explorations for Safety-Related Foundations, specifies the need for at least one boring underneath each projected safety-related structure or 1 boring for each 900 m^2. The footprint of the main containment shaft and the above ground surrounding structures is about 1 Ha (10,000 m^2). This implies that at least 10 borings would be required for the site investigation. RG 1.132 (Reference 8.64) indicates that the boring depth for soil-structure interaction studies should go past "the maximum required depth for engineering purposes." If bedrock is encountered, then the boring should penetrate past zones of weakness that could affect foundation performance and extend at least 6 m into sound rock. For the BWRX-300, the maximum required depth for engineering purposes d_{max} is set at approximately 120 m, a depth that is the greater than the following:

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

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

The extent and detail of the site investigation depend on the encountered subsurface conditions. Table 3-1 lists the expected types and amounts of tests that are required to properly characterize

site conditions. A boring and geophysical exploration layout is given on Figure 3-1. Table 3-2 lists the recommended borings and their purpose. A minimum of 21 boring locations is anticipated within the BWRX-300 site investigation program which exceeds the minimum of 10 borings based on recommendations in RG 1.132, Appendix D (Reference 8.64). The increase in the number of borings is to ensure adequate characterization of subsurface properties under and around the deeply embedded RB structure. Previously investigated sites may have information that cover a wide area. In such cases, only limited and targeted additional exploration points may be required.

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

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

Table 3-2: Anticipated Boring Program

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

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

3.1.2 Laboratory Testing Program

A laboratory testing program is performed on soil and rock samples collected from the site investigation program in accordance with the regulatory guidance of RG 1.138 (Reference 8.65) to obtain data for the analysis and design of the BWRX-300 RB. The scope and extent of the BWRX-300 laboratory testing program address the specific requirements of deeply embedded BWRX-300 design that requires a reliable set of data from laboratory tests for developing geotechnical inputs characterizing the properties of each subgrade material present at the site. Testing to estimate parameters for appropriate soil and rock discontinuities should be included using the methods identified in RG 1.138 (Reference 8.65). Testing of artificial interfaces for the foundation materials or use of adjacent material properties and sensitivity analyses, as described in Section 4.3.1.1, Interfaces Between the Structures and Subgrade Media, can be considered.

A laboratory testing program is implemented that depends on the site-specific subsurface conditions, the specific analysis requirements, and the need for sufficient data to adequately characterize variations in subsurface material properties. A sufficient number of laboratory tests are performed to minimize the uncertainties in the design related to these geotechnical input parameters by providing reliable estimates for the statistical parameters (mean and standard deviation values) of the measured material properties. The systematic (bias) errors are minimized by a carefully executed equipment calibration and sample management programs. Estimates of measuring bias are developed based on comparisons of measurements of physical parameters obtained from different types of subsurface material property tests.

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

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

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

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

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

3.1.3 Characterization of Rock Mass Properties

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

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

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

- over-excavation and backfilling;

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

Additionally, the existing groundwater conditions and the potential control of seepage through cavities, fracture zones, joints, bedding planes, and discontinuities is considered. Seepage control may include slurry walls, grouting prior to excavation, grouting during the excavation, freezing, drains, dewatering wells, sumps and other methods. The existing groundwater conditions and the impact from potential deterioration of shear strength or a reduction in rock mass quality is based on the results of piezometers and water-pressure tests described in the Site Investigation Program in Section 3.1.1.

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

To adequately assess and consider weak zones in rock masses, RG 1.132 (Reference 8.64) and NUREG/CR-5738 (Reference 8.2) provide guidance on logging and characterizing rock materials. Frequently, optical and acoustic televiewers (OTV/ATV) are used in conjunction with oriented or classical rock coring methods to map the depths, spacing, orientations, aperture, and other characteristics of the discontinuities. The geotechnical borings, wells, and borehole televiewer locations identified in Table 3-1, Site Investigation for the BWRX-300, are intended to collect rock data, groundwater information, and samples for laboratory testing to classify the rock mass. The type of information and testing required for the rock mass will depend on the specific subgrade conditions as well as the rock mass classification selected for the site. Inclined borings may be used to properly characterize the orientation of near vertical discontinuities.

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

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

RMR may be estimated following the parameters and ratings established by Bieniawski (1976, 1989, Reference 8.11). In order to use the RMR system, a rock mass is divided into different structural units defined by changes in rock type or major changes within a rock type, such as faults, fracture zones, or the spacing of discontinuities that may cause a change in the rock mass behavior. The RMR then considers semi-quantitative parameters for each structural region, which include the strength of the intact rock, RQD, the spacing of discontinuities, the condition of the discontinuities, the groundwater conditions, and the orientation of the discontinuities. Estimates of these parameters would be based on the data from the Site Investigation and Laboratory Testing programs. Even though GSI is now commonly used directly without an estimate based on RMR, RMR is retained because previous studies have indicated better estimates using RMR for the rock mass deformation modulus of moderate to strong rock masses (Galera et al., 2007, Reference 8.12). Comparisons can be made to in-situ measurements of rock modulus identified in Sections 3.1.1 and 5.2.1.2.

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

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

$$\mathrm{GSI} = 1.5\mathrm{JCond}_{89} + \frac{\mathrm{RQD}}{2} \qquad (3\text{-}1)$$

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

As described in RG 1.132 (Reference 8.64), characterization of the shear strength for planar discontinuities, such as bedding planes, faults, fracture zones, joints, and shear zones typically include laboratory testing of subsurface discontinuities recovered from samples (e.g., direct shear and triaxial compressive strength tests), tests on saw cut portions of recovered rock cores to determine base friction angles, or, less commonly, in-situ tests of the discontinuities under specific loading conditions (e.g., in-situ direct shear tests). In-situ direct shear tests at nearby outcrops or in shallow rock excavations can be considered. Because the most common method is testing

recovered subsurface samples, empirical corrections are required for surface roughness, intact surface strength, and the scale of the tested sample (e.g., Barton-Bandis criterion). When empirical corrections are used, the Site Investigation Program will need to determine estimates of appropriate parameters (e.g., the joint roughness coefficient [JRC] and joint wall compressive strength [JCS] for the Barton-Bandis criterion [ISRM, 1978]).

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

Cavities in the rock mass from karst or dissolution may decrease the effective rock mass modulus and create a highly variable interface between the rock and overburden. The presence of cavities should be identified during the subsurface investigation. Consistent with RG 1.132 (Reference 8.64), the spacing and depth of investigation locations should be reduced to detect the anticipated features.

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

3.2 Construction Inspection and Testing Program

3.2.1 Excavation and Foundation Inspections and Testing

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

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

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

- Rock material properties, such as rock type, color, particle size, hardness, and strength.

- Rock mass properties, such as rock structure, shear strength, deformation modulus, hydraulic conductivity, and attitude.

The additional geotechnical borings and borehole televiewer locations at the perimeter of the BWRX-300 RB shaft are intended to provide compensatory data if there is limited access for the excavation and foundation inspections and testing programs due to excavation support, rock reinforcement, surface treatments, and/or waterproofing. This additional data is intended to provide sufficient information to characterize inaccessible portions of the excavation.

When the rock excavation is accessible, additional in-situ testing can also be completed for rock mass properties such as shear strength and deformation modulus. These tests may include larger-scale plate loading of the in-situ rock mass in the excavation or in-situ direct shear tests on discontinuities exposed by the excavation using appropriate methods from RG 1.132 (Reference 8.64). Overcoring with a borehole deformation gauge in the rock excavation can also be implemented to confirm the in-situ stress field at some sites.

3.2.2 Building Structure Construction Inspections and Testing

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

- The visual surface inspection acceptance criteria that include quantitative limits for the appearance of leaching or chemical attack, pop outs or surface voids, scaling, spalling, corrosion staining, settlements, and cracks.
- ACI 349.3R guidance (Reference 8.18), which is recommended by ASME XI Rules for Inservice Inspection of NPP Components, Subsection IWL for visual inspections of exposed surfaces. ACI 349.3R requires that accessible concrete surfaces do not have voids greater than 2 inches; scaling is limited to 8 inches in diameter and 0.75 inches in depth; and cracks are limited to widths of 0.04 inches or smaller.
- ASME XI, Subsection IWL 1220 (b) and (d) exempts concrete surfaces that are covered by a liner or adjacent to a foundation or backfill from detailed visual inspections.
- Concrete surfaces exposed to soil, backfill, or groundwater are evaluated to determine susceptibility of the concrete to deterioration and the ability to perform the intended design function under conditions anticipated until the structure no longer is required to fulfil its intended design function. The evaluation includes the following:
    a) Existing subgrade conditions, including groundwater presence, chemistry, and dynamics; aggressive below-grade environment, or other plant-specific conditions that could cause accelerated aging and degradation.
    b) Existing or potential concrete degradation mechanisms, including, but not limited to, aggressive chemical attack, erosion and cavitation, corrosion of embedded steel, freeze-thaw, settlement, leaching of calcium hydroxide, reaction with aggregates, increase in permeability or porosity, and combined effects.

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

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

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

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

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

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

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

3.4 Field Instrumentation Plan

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

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

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

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

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

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

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

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

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

1. Piezometers to measure pore pressure distribution. Vibrating wire piezometers are preferred for this purpose as they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis. Piezometers should be screened at elevations that are representative of the site-specific hydrogeologic conditions.
2. Settlement monuments placed directly on concrete, preferably on the corners of the structures at grade that are accessible with conventional surveying equipment.
3. Settlement sensors and extensometers used for settlement prone soils or deformation prone rock masses.
4. Earth pressure sensors to monitor vertical and lateral pressure along the walls of the shaft.

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

The need for direct monitoring of interfaces will be a site-specific determination depending on their importance and the specific subgrade conditions.

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

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

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

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

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

4.3.1 Interface Models

4.3.1.1 Interfaces Between the Structures and the Subgrade Media

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

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

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

The parameters of the slipping spring can be taken from the material set of the adjacent soil/rock elements based on the results from the Site Investigation and Laboratory Testing programs as described in Section 3.1. A strength reduction factor can be used to adjust the spring stiffness based on the roughness of interaction and soil/rock residual strength when the sliding occurs. It is also possible to assign strength properties to interface elements based on direct measurements. If planar geosynthetic products are used during construction of the wall, shear properties are assigned to the interface elements representative of shear properties at geosynthetic/soil interfaces.

As is the case for soil and rock material constitutive models, the use of complex modeling capabilities for modeling interfaces introduces the challenge of identifying adequate input physical parameters. To address the uncertainties in these input parameters in a conservative manner, the analysis may be conducted using bounding limits for the rheologic elastoplastic models assigned

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

4.3.4 Analysis Staging Approach

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

4.3.4.1 Site Characterization

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

4.3.4.2 Excavation

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

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

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

below grade RB shaft exterior wall. The results obtained from the contact spring elements serve to:

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

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

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

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

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

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

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

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

5.1.2 Soil-Structure Interaction Modeling Assumptions

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

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

4) The static lateral pressures on the RB shaft due to the weight of ~~rock is assumed~~ self-supporting rock, i.e., excavated rock that does not require ~~no~~ lateral support ~~is required of the excavated rock~~ can be neglected.

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

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

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

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

The approaches described in Section 5.2.1.2 to calculate the equivalent linear properties of rock are applicable to structures that are relatively large compared to the block size of the rock mass and assumes the closely spaced discontinuities have similar characteristics where isotropic behavior of the rock mass is valid. For site-specific conditions where rock mass behavior may be highly anisotropic, adjusted rock Young's elastic modulus and Poisson's ratio are used as input for the design analysis to obtain conservative design demands that include adequate margins to envelope the effects of possible anisotropic behavior of rock masses. The equivalent linear properties of the rock mass may be adjusted based on the results of the FIA performed using constitutive models that can capture the important aspects of the anisotropic and non-linear behavior of the rock mass as described in Section 4.2.2. The adequacy of the design margins is evaluated based on the levels of uncertainty present in the determination and modeling of the site-specific subgrade conditions.

When the discontinuity spacing is large compared to the dimensions of the excavation, the potential for unstable blocks or wedges and swelling or squeezing rock units need to be evaluated. The evaluation of the potential loads on the RB from rock blocks and wedges may be completed using results obtained from:

- FIA models that include the rock discontinuities, as the ones described in Section 4.3.1.2, and/or

- simple static or pseudostatic force limit equilibrium analysis models as the one shown in Figure 5-1. ~~A simple example of a model for force equilibrium analysis of rock stability is provided in Section 5.1.4.3.~~

Equivalent lateral coefficients may be calculated using upper bound lateral pressure estimates obtained from the FIA or limit pseudostatic force equilibrium analyses. Equation (5-14) is then used to calculate Poisson's ratio values for the rock mass having the potential for unstable blocks or wedges. These Poisson's ratios and upper bound unit weights are assigned to the rock mass to ensure the design adequately addresses the potential for unstable rock blocks or wedges.

Strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness will frequently be self-supporting even if some reinforcement is required to ensure a safe excavation. Typically, rock masses will yield slightly during construction – even with well-placed reinforcement – and arching will reduce the lateral loads except in highly fractured, weak, swelling, or squeezing rocks. ~~Joints and other weak planes may create isolated blocks that are unstable; however, these blocks are not typically large relative to the area of the structure and would be unlikely to produce significant loads on the exterior of the structure compared to other loads (e.g., hydrostatic). These blocks would also not be able to create a cascading failure once the structure is in place.~~

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

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

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

The effects of secondary non-linearity induced in the soil and rock by the structural vibration are neglected because in general, the structural vibration induces plastic deformations of the soil and dissipation of energy in the SSI system that reduces the structural response as shown in Reference 8.27 and Reference 8.28. On the other hand, the secondary non-linearity of subgrade

materials may amplify the magnitude of the dynamic lateral pressures. The presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones within the rock mass may also affect the stability of individual blocks or the rock mass during an earthquake that can potentially amplify the seismic rock pressure loads. Section 5.3.11 describes the approach used to evaluate the effects of subgrade materials non-linearity on the seismic response and design BWRX-300 RB when it is constructed at sites characterized with a high non-linear behavior of the subgrade materials and high seismicity. To account for possible amplifications of dynamic earth pressures on the RB shaft due to rock discontinuities, fracture and weak zones, the dynamic stiffness and unit weight properties of the rock masses may be adjusted using the same approaches as the ones used for adjusting the static rock properties.

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

5.1.3 Design Earth Pressure Load Validation

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

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

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

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

The validation of soil and rock pressure loads may consider the subgrade improvements like consolidation grouting, rock reinforcement, and soil support made during the construction. However, these improvements are typically considered only as initial ground support that is

separate from the permanent ground support system because these types of reinforcements and any surface protection will be inaccessible for monitoring and repair after the construction. Therefore, unimproved soil and rock conditions are considered due to the uncertainty in:

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

This additional rock load on the RB shaft wall may be considered uniform with contact grouting to avoid stress concentration or point load associated with the block or wedge that is reinforced to stabilize the rock excavation. The evaluation of these rock pressure loads assumes that the excavation has reached stability with initial rock support and that the liner will accept 100 percent of the initial rock support as it relaxes over the lifetime of the structure. The~~se~~ initial rock support loads should be conservative because rock loads in stressed rock masses are typically not following (e.g., they ~~are not independent of displacement and~~ typically reduce with displacement due to arching). The notable exception would be due to the presence of hydrostatic loads and swelling or squeezing rock displacements that are time dependent and can ~~may~~ continue to apply a large load with continued displacement. When pressures from swelling or squeezing rock displacements create the potential for loads on the RB shaft, the RB shaft can be designed to take the full pressure, a compressible material can be placed between the RB shaft and rock to reduce the pressures, or a scheduled construction delay can be added to allow the deformation to occur and reduce the pressures. If the degradation of initial support for large rock blocks potentially creates unacceptable high pressures, other options can include overexcavating and backfilling the rock block to reduce the potential pressures, the use of degradation resistance rock reinforcement to permanently support the rock block, or changes in the BWRX-300 location to improve the relative position of the rock block and reduce the potential pressures on the RB shaft.

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

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

The pressure load validation FIA uses the constitutive models described in Section 4.2 to represent the non-linear response of soil and rock subgrade materials, and the models described in Section 4.3.1 to represent the response at interfaces including the interfaces of RB structure with the surrounding subgrade. Because the intent of the FIA is to calculate best estimates of the soil

and rock pressure loads, constitutive and interface models are developed using best estimate soil and rock properties obtained from the results of site investigation and laboratory testing programs described in Section 3.1. The stiffness of the RB structure in the FIA models is calculated per the governing design codes. Conservative design values obtained from the literature can also be used for certain input parameters.

A best estimate soil and rock pressure profile on the RB shaft is developed as an envelope of all maximum lateral pressure values calculated by the non-linear FIA of all analyzed post-construction stages and scenarios. This lateral pressure profile is compared to the lateral pressure profile developed from the results of the linear elastic 1-g design analysis to confirm the equivalent linear elastic model provides adequately conservative loads for the structural design. Soil and rock design pressure margins are calculated based upon the minimum values and the distribution of the ratio between the design soil and rock pressures obtained from the 1-g linear elastic analysis and the best estimate pressures obtained from the non-linear FIA. If the values of the calculated soil and rock design load margins are below the values deemed adequate to address the uncertainties and variations of subgrade properties, the rock mass weight or the equivalent linear soil and rock stiffness properties used for the 1-g design analysis are adjusted following the recommendations in Section 5.1.2. Adequate values of the soil and rock design load margins are established based on the uncertainties and variability of soil and rock properties used as input for the non-linear FI~~E~~A and the significance of the non-linear and anisotropic response of subgrade materials on the soil and rock pressure demands. Since the earth pressure load is the most important site-related load affecting the structural integrity of the deeply embedded RB structure, the validation of the earth pressures often is sufficient to address the uncertainties in the important site subgrade parameters that can affect the design of the RB structure. For sites where the consideration of earth pressures alone cannot completely address all uncertainties related to the subgrade stiffness parameters, the adequacy of the design may be demonstrated by comparing the non-linear FIA and the design 1-g SSI analysis displacement results at the interfaces of the RB structure with the surrounding soil and rock. Alternatively, the FIA calculated displacements may be applied to the RB structural design analysis model as boundary conditions at the soil-structure interfaces to calculate stress responses of selected major RB structural members that are in line with the FIA calculated subgrade response. These FIA compatible structural stress results are then compared to the corresponding results of the design 1-g SSI analysis to demonstrate that the design adequately address the uncertainties related to the determination of subgrade stiffness parameters.

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

5.1.4 Probabilistic Earth Pressure Analyses

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

below-grade exterior walls and using them to determine the probability for the earth pressures on the RB shaft to exceed the design pressure loads calculated by the 1-g SSI analysis.

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

- Parameter uncertainties related to natural randomness ~~and uncertainties in measurements of mechanical properties~~ of in-situ subgrade materials, errors in their measurements and uncertainties related to development of site subgrade parameters; and
- Model uncertainties related to the simplifying modeling assumptions discussed in Section 5.1.2~~models used for earth pressure calculations~~.

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

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

The model uncertainty that represents the uncertainty related to the model's ability to accurately predict the soil and rock pressures is manifested as a bias error in the earth pressure calculations. In general, the model uncreate is reduced by using more sophisticated models and an increasing number of model parameters. On the other hand, the increasing number of parameters used in the sophisticated models increases the parameter uncertainty and may reduce the overall confidence in the calculated soil pressure results. The model uncertainty is approached by means of considering different models that utilize fewer input parameters resulting in discrete probability distributions that are combined as described in Subsection 5.1.4.4.

As noted in Section 5.1.3, the structural stress results compatible to the deformations at the RB interfaces with the surrounding soil and rock may be used to address the uncertainties related to the modeling of subgrade stiffness parameters. For sites where probability calculations may be required to demonstrate that the available design stress margins are adequate, the probability of the structural stresses responses to exceed the stress demands used for the design of the RB structure may be obtained from probabilistic analyses performed using simplified FIA models and following the approaches described in this section.

Table 5-1: Models for Probabilistic Earth Pressure Analyses

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

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

a. at-rest condition representing essentially no movement of the structure relative to the surrounding subgrade;

b. active condition when the structure moves away from the surrounding subgrade; and

c. passive condition when the structure moves towards the surrounding subgrade.

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

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

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

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

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

α Menard's correction factor.

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

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

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

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

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

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

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

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

where *OCR* is the over-consolidation ratio.

5.2.1.2 Rock Mass Equivalent Linear Properties

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

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

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

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

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

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

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:

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

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:

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

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:

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

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

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

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

Depending on the site conditions, in-situ testing with rock pressure meters (dilatometers) or borehole jacks may also be considered to estimate E_{st} of the rock masses as identified in Section 3.1.1.

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

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