ENCLOSURE 2

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BWRX-300 Advanced Civil Construction and Design Approach

Non-Proprietary Information

3.1.1 Site Investigation Program

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

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

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

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

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

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

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

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

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

requires a reliable set of data from laboratory tests for developing geotechnical inputs characterizing the properties of each subgrade material present at the site.

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

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

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

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

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

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

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

3.1.3 Characterization of Rock Mass Properties

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

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

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

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

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

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

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

To adequately assess and consider weak zones in rock masses, RG 1.132 and NUREG/CR-5738 (Reference 8.2) provide guidance on geologic mapping, logging and characterizing rock materials.

Geologic mapping and geotechnical borings described in Section 3.1.1 are used to characterize the intact rock, rock discontinuities, and the rock mass. Frequently, optical and acoustic televiewers (OTV/ATV) are used in conjunction with geologic mapping and oriented or classical rock coring methods to map the depths, orientations, aperture, and other characteristics of the discontinuities. The type of information and testing required for the rock mass will depend on the specific subgrade conditions as well as the rock mass classification selected for the site. When other data or geologic mapping indicates near vertical discontinuities may be present, ~~I~~inclined borings may be used to properly characterize the orientation and strength of near vertical discontinuities.

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

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

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

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

GSI may also be estimated semi-quantitatively for rock masses where block sliding, and rotation is the primary means of failure. This semi-quantitative method was developed for use when a qualified and experienced geologist or engineering geologist does not observe the rock mass and is recommended to supplement and not replace the qualitative estimates by a qualified and experienced professional. The quantitative input includes the RQD and the joint condition

($JCond_{89}$). Similar to the GSI, the $JCond_{89}$ value is based on a qualitative evaluation of the discontinuity surface and other features, including persistence, aperture, roughness, infilling, and weathering (Hoek et al., 2013, Reference 8.14). Alternatively, the $JCond_{89}$ may be estimated from a reduced set of estimates known as the joint roughness number (Jr) and joint alteration number (Ja) following Hoek et al. (2013, Reference 8.14). The semi-quantitative relationships for GSI and $JCond_{89}$ from Hoek et al. (2013) are provided below:

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

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

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

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

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

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

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

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

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

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

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

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

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

3.4 Field Instrumentation Plan

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

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

- Measuring the rate of heave during excavation, especially at the end of excavation and at the bottom center and edges of the shaft.
- Measuring the rate of lateral displacement of excavation walls, throughout its depth, during and at end of excavation.
- Measuring the distribution of pore pressures around and below the RB shaft.

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

- Interface modeling described in Section 4.3.1; allows the introduction of the response and failure criteria between geometric zones; this feature is necessary to analyze faults, rock slip surfaces, or other discontinuities around the structure. The interface modeling has non-linear modeling capabilities.
- Interface modeling between soil/rock and structure described in Section 4.3.1; which is necessary to incorporate interaction between concrete and soil/rock via friction, accounting for the selected construction method and final configurations at the structure-soil/rock contacts. Non-linear behavior and separation are parts of the capability of this feature.
- Structure modeling, which may be limited to the main civil/structural components of the RB: main walls, floors, pools, and auxiliary structures.
- Soil/rock anchors and geogrids, which are used to simulate stabilization of the excavation and any associated potential failure surfaces.
- Fluid-soil interaction, which may be considered if the modeling the position of a static, horizontal groundwater table is not sufficient for the complexities in the design and construction of the BWRX-300 RB. Pore pressures are dependent on the permeability of the subsurface media, the hydrogeologic configuration, and the dewatering strategies for construction and operation.
- Staging analysis with time-dependent capabilities, which enables modeling the interaction of the structure and surrounding subgrade from excavation, through construction, loading and final operation. The model is capable of following stress/strain response as stress regimen changes from unloading during excavation to reloading after construction and during operation.

4.2 Subgrade Material Constitutive Models

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

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

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

4.3 Non-Linear Foundation Interface Analysis Approach

The FIA addresses the following aspects:

- Interface modeling, described in Section 4.3.1, including both (a) contacts between structure and soil/rock, and (b) fault or joint planes or interfaces between bedding units in a geologic formation.
- Structural modeling of the main civil/structural components of the BWRX-300 and auxiliary facilities, described in Section 4.3.2, along with varying live and dead loads throughout the construction process.
- Fluid Soil Interaction, described in Section 4.3.3, to capture an adequate distribution of the space and time variation of pore pressures.
- BWRX-300 life stages: siting, excavation, construction, loading, and operation described in Section 4.3.4.

4.3.1 Interface Models

4.3.1.1 Interfaces Between the Structures and the Subgrade Media

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

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

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

The parameters of the slipping spring can be taken from the material set of the adjacent soil/rock elements or strength tests on natural and artificial discontinuities from the site investigation, laboratory testing program and characterization programs as described in Sections 3.1.2 and 3.1.3. The development of the interface parameters should be consistent with the limitations and modeling guidance of the software and interface model used for the nonlinear FIA. A strength reduction factor can be used to adjust the spring stiffness based on the roughness of interaction and soil/rock residual strength when the sliding occurs. It is also possible to assign strength properties to interface elements based on direct measurements. If planar geosynthetic products are used during construction of the wall, shear properties are assigned to the interface elements representative of shear properties at geosynthetic/soil interfaces.

As is the case for soil and rock material constitutive models, the use of complex modeling capabilities for modeling interfaces introduces the challenge of identifying adequate input physical parameters. To address the uncertainties in these input parameters~~ in a conservative manner~~, ~~, the analysis may be conducted using bounding limits for the rheologic elastoplastic models assigned to the interface. One bounding scenario is a continuous connection case for which high stiffnesses (k) and soil equivalent failure criteria (ϕ, c) are assigned to the interface. S~~sensitivity ~~evaluations~~ analyses may be conducted ~~assuming lower friction and variations of the interface stiffness~~by adjusting ~~initial~~ spring stiffness and shear strength directly or through strength reduction factors. These types of analyses provide insight to understand the uncertainty introduced by interfaces in the stress distribution and deformation response of the structure.

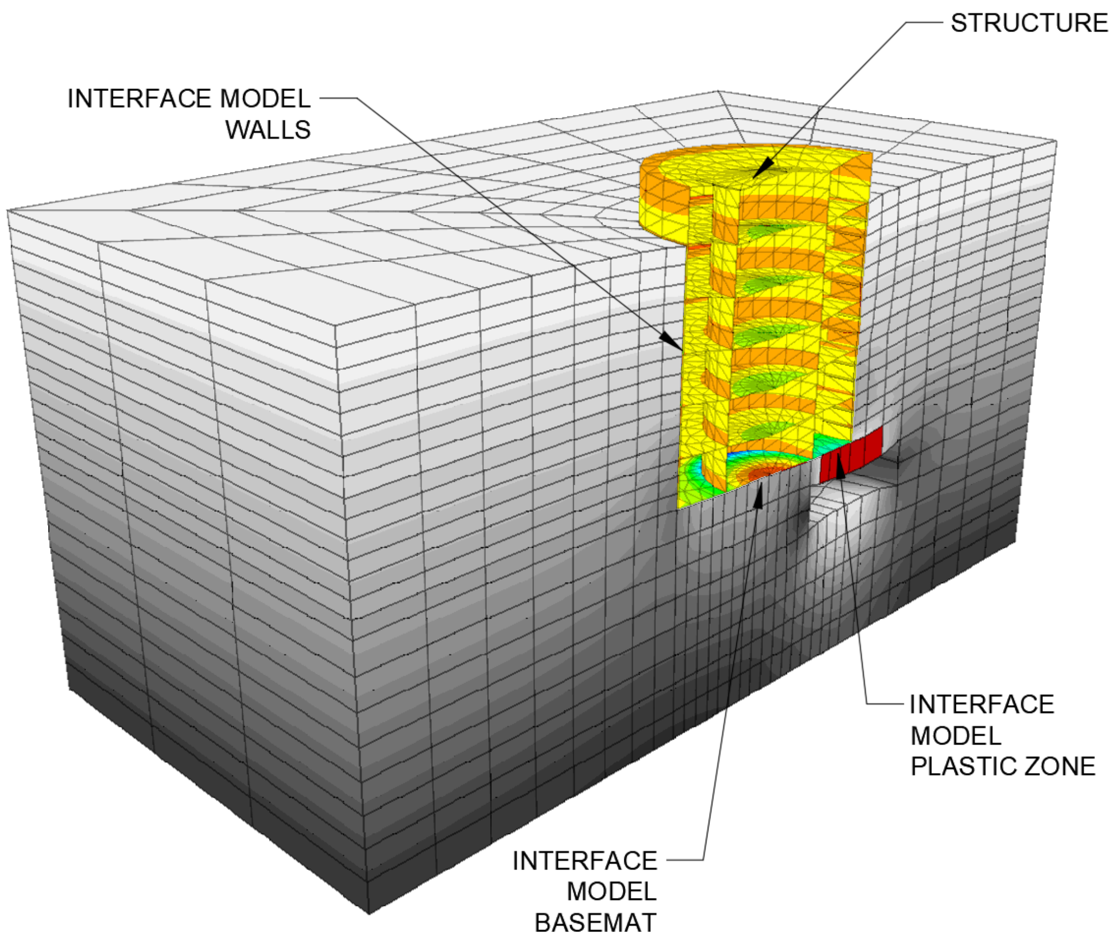

Figure 4-1: Location of Interfaces between Soil and Structure

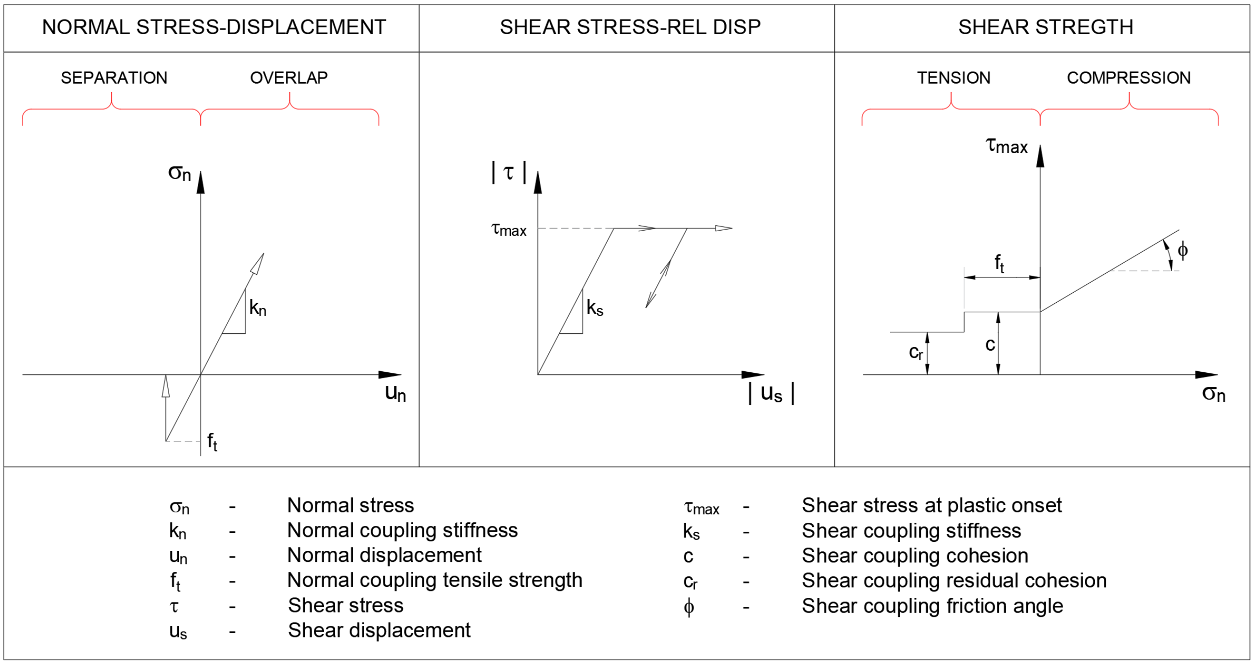

Figure 4-2: Interface Rheologic Modeling

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

The embedment depth of the BWRX-300 allows the possibility that soil rock interfaces, bedding interfaces, and other joints (Figure 4-3) may be in contact with the sides and base of the structure. These features may have planar or irregular configuration, and may be horizontal or with dipping, and even striking angles with respect to the position of the structure. The non-linearity and behavior of the joints are analyzed throughout the life stages of the reactor. These interfaces are modeled using similar interface modeling approaches as described in Section 4.3.1.1. The strength properties assigned to the interface elements along a rock discontinuity, i.e. bedding, are obtained from laboratory ~~or field~~ testing data described in Section 3.1.~~3~~2. ~~If~~When multiple strength tests are performed for rock discontinuities, ~~T~~the weakest strength parameters ~~representing the slipping may also be estimated based on the properties of the weakest interface material~~can be used for the interface elements or sensitivity analyses may be completed similar to Section 4.3.1.1. Strength reduction factors may be used to adjust the spring stiffness and shear strength based on the ~~roughness and residual~~ strength of the interface where~~n~~ the sliding occurs.

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

4.3.4 Analysis Staging Approach

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

4.3.4.1 Site Characterization

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

4.3.4.2 Excavation

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

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

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

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

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

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

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

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

5.1.1 FE Model of RB Structure

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

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

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

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

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

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

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

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 for subgrade modeling used for the design SSI analyses performed following the SASSI methodology:

1) The properties of the subgrade materials are assumed to be isotropic and linear elastic;
2) The non-linearities at soil-structure interfaces are neglected;

3) The rock mass is assumed continuous and the presence of cavities, fracture zones, joints, bedding planes, discontinuities and other weak zones is neglected;

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

The effects of secondary non-linearity induced in the soil and rock by the structural vibration are neglected because in general, the structural vibration induces plastic deformations of the soil and dissipation of energy in the SSI system that reduces the structural response as shown in protection will be inaccessible for monitoring and repair after the construction. Therefore, unimproved soil and rock conditions are considered due to the uncertainty in:

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

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

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

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

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

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

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

5.1.4 Probabilistic Earth Pressure Analyses

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

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

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

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

- Statistical error in the mean that can be reduced with increasing the sample size and number of measurements and tests being performed
- Bias in sampling and measurement procedures that is corrected by means of correction techniques/algorithms

- Bias introduced by the method used for development of subgrade parameters that is addressed by considering different approaches and empirical equations to calculate discrete probability distributions that are then combined as described in Subsection 5.1.4.4.

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

5.1.4.1 First Order Second Moment Method

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

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

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 kth measured data point of the parameter x_i.

- a FE model or a finite difference model.

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

Table 5-1: Models for Probabilistic Earth Pressure Analyses

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

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

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

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

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

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

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

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

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