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GEOTECHNICAL PARAMETERS FOR DESIGN OF DEEPLY EMBEDDED SMALL MODULAR REACTORS

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ABSTRACT

This paper presents approaches for development of geotechnical parameters for standard and site-specific design of a deeply embedded Small Modular Reactor (SMR). The interaction of the deeply embedded SMR structure with the surrounding soil and rock is important for their response under seismic, static, and thermal design loads. Therefore, the properties of the in-situ soil and rock materials have a significant effect on the design of these SMRs due to their deep embedment and smaller size. A set of eight profiles of equivalent linear soil and rock static and dynamic properties are presented for use as input for standard design of SMRs. Recommendations are provided for development of equivalent linear properties of soil and rock materials from data collected from field and laboratory tests that ensure the site-specific design of the deeply embedded SMR adequately addresses variations and uncertainties in the subsurface conditions. Approaches for developing parameters for characterizing rock masses and modelling the subgrade non-linear and anisotropic behaviour are also provided for use as input for non-linear Foundation Interface Analyses (FIA) performed to ensure the stability of subgrade materials and foundations surrounding the deeply embedded SMR.

STANDARD DESIGN GEOTECHNICAL PARAMETERS

A set of eight profiles of in-situ soil and rock static and dynamic properties are developed for use as input for standard design from a database of measured dynamic properties of subsurface materials by grouping and averaging the properties of sites with similar surficial geology. These generic subgrade profiles reflect site conditions with soft surficial residual soils being removed from the site prior to construction, which is necessary to provide adequate access to the site and conditions necessary for adequate support and operation of heavy cranes and other construction equipment.

The generic profiles provide a realistic representation of a wide range of geotechnical conditions existing at sites across North America that are suitable for deployment of SMRs. Each profile is defined in terms of average measured shear wave velocity of the top 30 meters of soil ($\bar{V}s_{30}$) and the depths to the geological base rock. For example, the generic profile 180-600 represents a generic site subgrade condition where the average measured small-strain shear wave velocity over the top 30 m of soil is $\bar{V}s_{30} = 180\ m/sec$ and the geological base-rock is located at depth approximately 600 m below the profile surface. The eight generic profiles represent a range of conditions varying from a deep medium-stiff soil Profile 180-600 to a shallow stiff soil/soft rock Profile 500-21 to a hard rock Profile 2032-30.

Figure 1 presents generic profiles of soil and rock dynamic properties, shear and compression wave velocities, damping and unit weight for use as input for seismic standard design of SMRs. The soil shear wave velocities and damping properties are compatible to the strains generated by design level earthquakes defined by Certified Design Response Spectra (CSDRS) (Todorovski at. al. 2013). These strain compatible properties are developed based on results of probabilistic site response analyses (SRA) performed on suites of randomized generic shear wave velocity profiles of measured subgrade properties (Todorovski at. al. 2013). The compression wave velocity profiles reflect saturated soil conditions with values of the Poisson's ratio (ν) of softer soil layers ranging from 0.48 to maximum of 0.49 to ensure the numerical stability of Soil-Structure Interaction (SSI) analysis results.

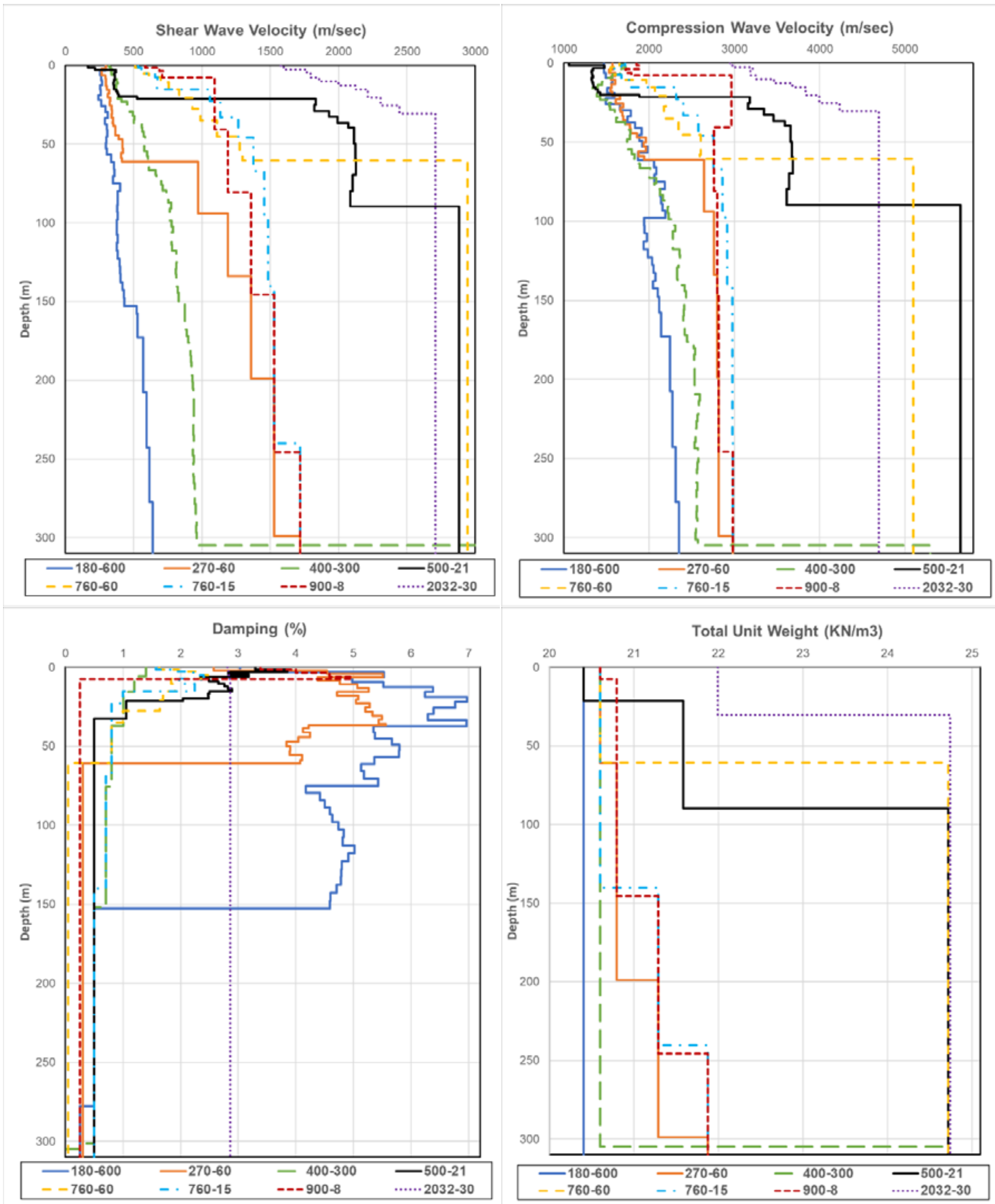

Figure 1. Generic Profiles of Subgrade Dynamic Properties

Table 1 provides a set of generic engineering properties for different types of subgrade materials at candidate nuclear sites. The table provides upper bound estimates of unit weights and lateral pressure coefficients with estimates of friction angles and lower bound base friction coefficients that produce a conservative standard design for deeply embedded SMR structures.

Table 1: Generic Design Parameters for Soil and Rock Materials

Soil Type	Unit Weight (kN/m^3)		Friction Angle (degree)	Lateral Pressure Coef.			Base Friction Coefficient
	Dry	Total		K_0	K_a	K_p	
Medium Soil	18.3	21.2	35	0.620	0.271	3.69	0.43
Firm Soil	20.4	22.5	40	0.568	0.217	4.60	0.50
Stiff Soil	23.0	24.1	45	0.518	0.172	5.82	0.58
Rock	25.0	25.0	N/A	0.429	N/A	5.82	0.60

The generic soil parameters in Table 1 reflect properties of cohesionless soil materials that adequately represent soil conditions at many candidate sites, which are comprised of gravels, sands, and low plasticity clays. Upper bound dry unit weights are provided for the different soil materials representing properties of well compacted silty sand and gravel materials that are characterized by the highest unit weights among those of other granular soils. The soil friction coefficients (ϕ_s) values in Table 1, that represent lower bound strength properties of compact to very dense granular soil materials, were used to calculate upper bound estimates of the soil lateral pressure coefficients.

At-rest lateral pressure coefficients for the soil materials are calculated using the following equation for normally consolidated materials (Lambe and Whitman, 1969; Equation 10.1):

$$K_0 = 1 - \sin\left(\phi_s\right) \tag{1}$$

The soil active (K_a) and passive (K_p) lateral pressure coefficients are calculated using the Rankine theory equations with a level grade:

$$K_a = \frac{1 - \sin(\phi_s)}{1 + \sin(\phi_s)} \qquad K_p = \frac{1 + \sin(\phi_s)}{1 - \sin(\phi_s)} \tag{2}$$

The soil active pressure coefficients are for calculations of lateral pressure demands on soil retaining walls that are associated with larger lateral deformations. The passive pressure coefficient values in Table 1 provide lower bound estimates of the lateral bearing pressure capacity of the subgrade materials. The at-rest coefficient (K_0) for rock in Table 1 is calculated from the following elastic theory equation using a value for rock Poisson's ratio $\nu_r = 0.3$:

$$K_0 = \frac{\nu_r}{1 - \nu_r} \tag{3}$$

The last column of Table 1 provides values for friction coefficients between the concrete base and different types of subgrade materials for standard design foundation stability evaluations. Based on common engineering practice, generic values for the friction coefficients are calculated as two thirds (2/3) of the soil friction angle, and a value of 0.60 is adopted for the friction coefficient between the concrete and rock. If a water proofing membrane is placed below the basemat, the standard design foundation stability evaluations can use a minimum value of the provided soil base friction angle and membrane coefficient of friction with concrete. A conservative value of 0.5 can be adopted for the membrane coefficient of friction is based on the results of testing of different water proofing materials.

Figure 2 presents generic profiles of soil and rock static elastic modulus, Poison's ratio, and dry unit weight for use as input in the standard design analysis to account for the interaction of the deeply embedded SMR with the surrounding subgrade materials under static and thermal design loads.

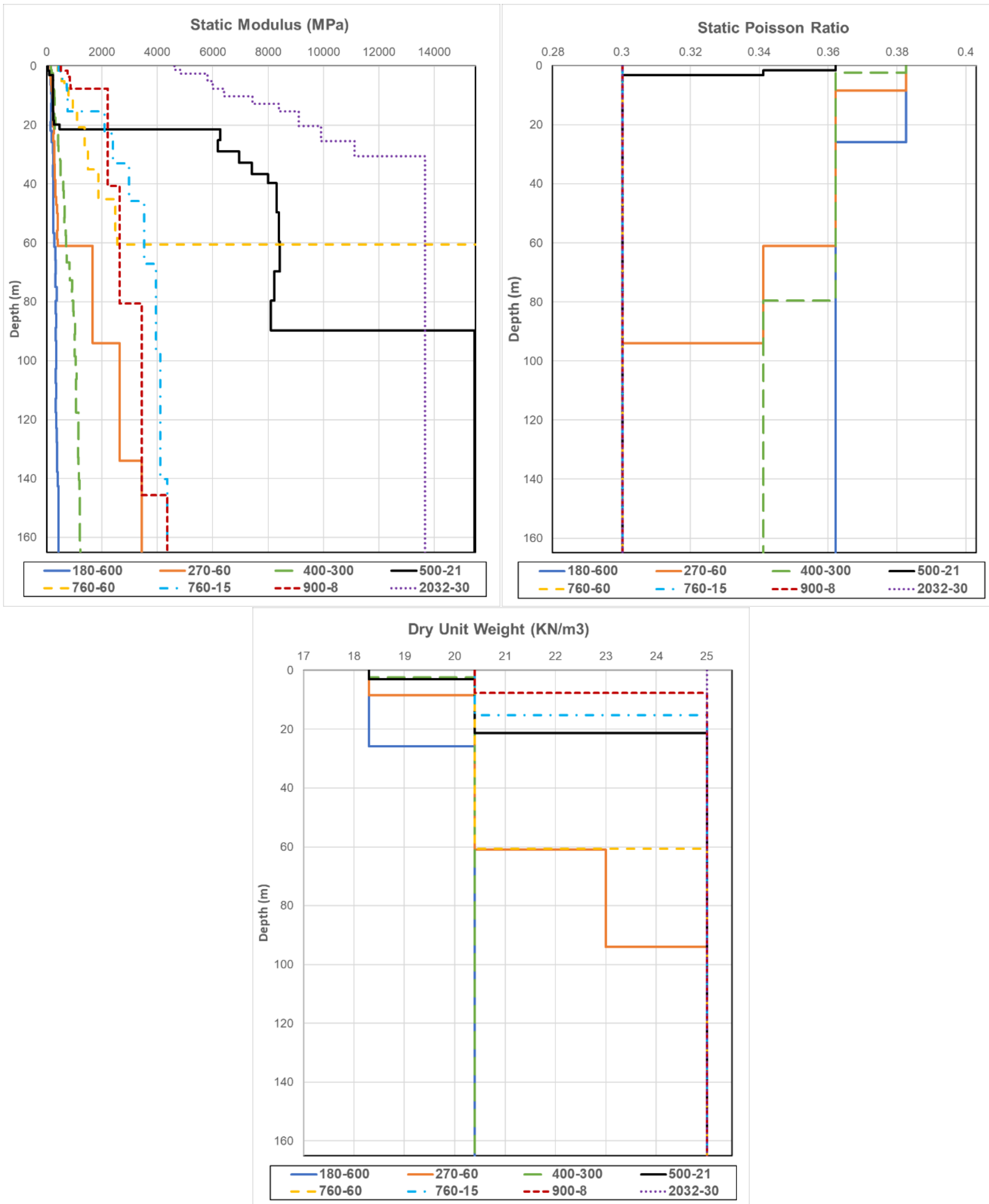

Figure 2. Generic Profiles of Subgrade Static Properties

The soil and rock parameters provided in Table 1 are conservatively correlated with the generic profiles of measured small-strain dynamic properties to develop the generic profiles in Figure 2 representing the variations of the soil and rock static properties as function of depth. Medium soil properties are assigned to layers with small-strain shear wave velocities lower than 350 m/sec. Firm and stiff soil properties are assigned to layers with small-strain shear wave velocities ranging from 350 m/sec to 760 m/sec and from 760 m/sec to 1000 m/sec, respectively. Layers with small-strain shear wave velocities larger than 1000 m/sec are assigned rock properties.

Elasticity theory is used to calculate values for the Poisson's ratio (ν_{st}) for the soil materials that are representative of at-rest lateral pressure conditions defined by upper bound estimates of the at-rest pressure coefficients (K_0):

$$\nu_{st} = \frac{K_0}{1 + K_0} \tag{4}$$

These Poisson ratio values together with upper bound soil dry unit weights yield conservative estimates of the static earth pressure loads for the standard design of deeply embedded SMR structures.

Static elastic modulus (E) values in Figure 2 are the other parameter representing the equivalent stiffness properties of the subgrade materials under long-term loads. The elastic modulus values are calculated from elasticity theory with stiffness degradation (FHWA, 2017; Equation 8.18):

$$E = D_E 2 \left(\frac{\mathrm{w}}{\mathrm{g}}\right) V_S^2 (1 + \nu_{st}) \tag{5}$$

where: $D_E = 0.28$ static stiffness degradation coefficient

V_S small-strain (measured) shear velocity of subgrade material

w is soil dry unit weight

$g = 9.81 \mathrm{m/sec}$ is the Earth's gravity constant

Considering anticipated strain levels in the subgrade materials corresponding to a bearing capacity factor of safety of 3, the value of static stiffness degradation coefficient $D_E = 0.28$ used in Equation (5) is obtained from the elastic modulus (E/E_0) degradation curve in FHWA NHI-16-072 (2017). This degradation curve is applicable for intact clay and uncemented sand materials and provides lower values of stiffness degradation for the rock that is similar to an undisturbed rock mass with a Geologic Strength Index (GSI) of 50. These lower bound elastic modulus values result in conservative estimates of the deformations and stresses demands on the deeply embedded SMR structures under long term static loads.

SITE-SPECIFIC DESIGN GEOTECHNICAL PARAMETERS

The site-specific design and safety evaluations of deeply embedded SMR requires a number of site-specific geotechnical parameters that serve as inputs for:

- the SRA used to define the ground motion amplifications and strain-compatible dynamic subgrade properties for site-specific seismic design,
- the non-linear FIA performed to demonstrate the stability of the subgrade materials and foundations surrounding the deeply embedded SMR,
- the static (1-g) gravity SSI analyses to simulate static earth pressure demands for the site-specific design of the deeply embedded SMR structure, and
- defining site-specific subgrade elastic moduli to account for the effects of subgrade stiffness on the response of the deeply embedded SMR under internal mechanical and thermal loads.

The flow chart in Figure 3 illustrates the relationship between the different subgrade parameters and analyses used for the structural design of deeply embedded SMR.

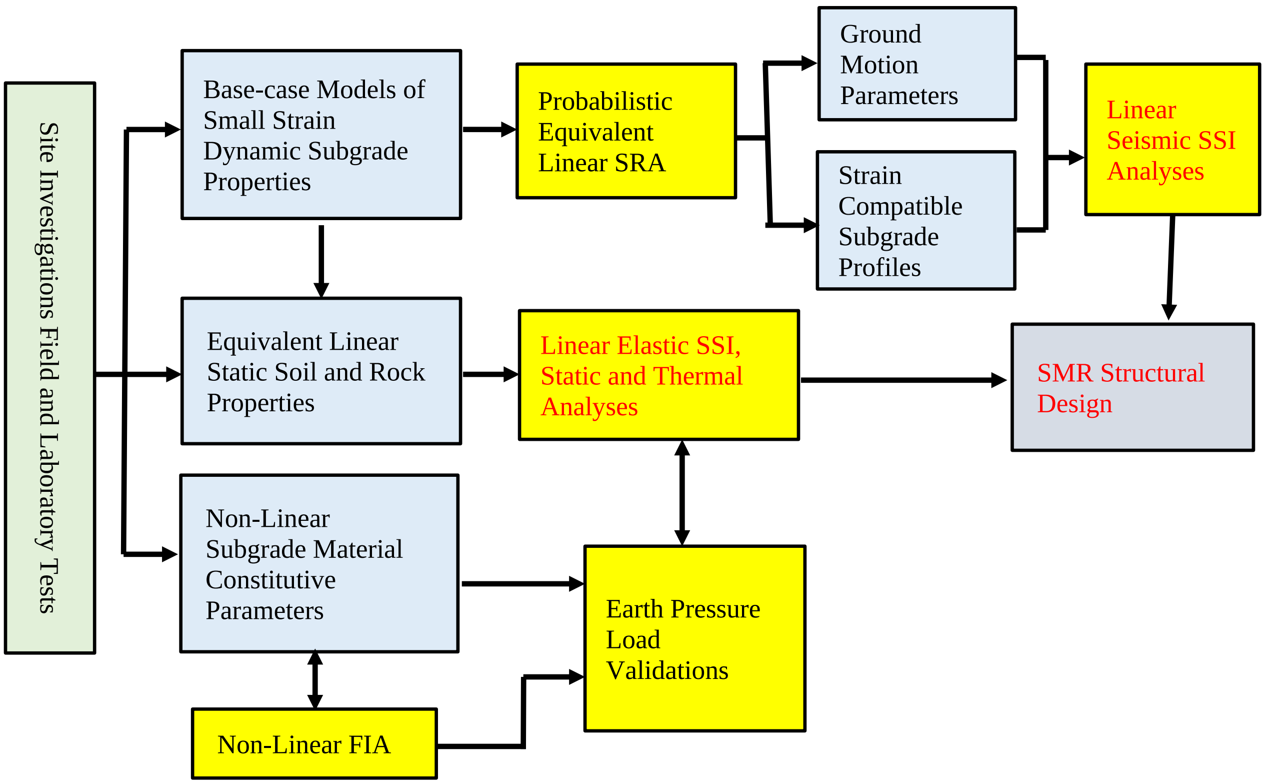

Figure 3. SMR Structural Design Process

For the subsurface conditions encounter at most of the candidate sites, approaches are employed for the construction of deeply embedded SMRs that optimize the cost and reduce the construction schedule by minimizing the amount of excavation and reducing the amount of engineered backfill (NEDO-33914 2022). Hence, the engineering properties and the spatial distribution of in-situ soil and rock materials are far more important for the stability and safety of the deeply embedded SMRs than for large nuclear power plants that are founded near the plant surface and, if embedded, are surrounded by large quantities of engineered backfill. The geotechnical parameters for the site-specific design of deeply embedded SMRs are established from the results of site investigation and subsurface material testing programs which extends beyond those that are necessary for large nuclear power plants.

The footprint dimensions of the SMRs are relatively small when compared to footprints of conventional nuclear plants. Therefore, the site investigation program covers a larger area than the footprint of the SMR with a denser spacing of boreholes that characterizes the variations and uncertainties in the subsurface conditions and provides the data to evaluate the stability of the subgrade materials around the deeply embedded SMR. If there are foundation in the vicinity of the deeply embedded SMR structure which can affect the stability of the SMR, the site investigation also covers the footprint of these foundations. The maximum depth of the investigation program is the anticipated depth where the vertical stress is expected to be less than 10 % of the in-situ condition.

Geologic and engineering parameters of rock masses are quantitatively characterized using empirical engineering and geomechanical rock mass classifications, such as the Rock Quality Designation (RQD) index, Rock Mass Rating (RMR) system, and GSI. These rock classifications are made considering a variety of parameter ratings 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 to evaluate the range of rock mass characteristics encountered during subsurface characterization and multiple classifications systems are considered to incorporate uncertainty in the parameter estimates. Estimates of RQD may are on recovered rock cores and confirmed using data from optical and acoustic televiewers or estimated from mapped or scanned surfaces based on the average number of discontinuities or a volumetric joint count (Hoek et al. 2013).

If a part of the SMR structure is embedded in rock, evaluations are performed to determine possible rock pressures on the SMR structure. Only strong rock without disadvantageous fracture zones, joints, bedding planes, discontinuities and other zones of weakness may be self-supporting without lateral support when excavated. The design can neglect the static lateral pressures on the SMR below grade walls due to the weight of self-supporting rock masses only if reinforcement is not required to ensure a safe excavation. Typically, even when reinforced, rock masses will yield slightly during construction, and arching will reduce the lateral loads except in highly fractured, weak, swelling, or squeezing rocks. Nevertheless, the SMR design considers the rock reinforcement as initial ground support because the rock reinforcements may be inaccessible after construction. The rock loads remaining after the initial ground support degrades are addressed by including the potential pressures from rock in the design based on the non-linear FIA.

The potential in-situ state of stress in the bedrock is evaluated to determine potential lateral rock pressure loads on the SMR below-grade walls. These evaluations include reviewing the state of stress in the crust as part of evaluating the tectonic framework and the potential for unrelieved stresses in bedrock near the site. A review of regional and 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 is used to assess the likelihood for increased horizontal stress in the bedrock. If results of these evaluations indicate high residual horizontal stresses in the rock mass, site-specific measurements of the in-situ state of stress in bedrock formations are collected as part of the geotechnical borings and borehole televiewer tests.

Discontinuities and other zones of weakness within the rock mass may result in instabilities of individual blocks or the rock mass when their orientation is disadvantageous and the spacing of discontinuities is sufficiently dense. The presence of discontinuities may also affect the load transfer from adjacent shallow foundations on the deeply embedded SMR. 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. Discontinuities and other zones of weakness in rock masses are assessed by geologic mapping and characterizing rock materials using data collected from geotechnical borings. Optical and acoustic televiewers can be 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 or other weak zones. The type of information and testing required for the rock mass depends on the specific subsurface conditions as well as the rock mass classification selected for the site. When there are indications of presence of near vertical discontinuities, inclined borings may be used to properly characterize the orientation and strength of near vertical discontinuities. Estimates of earth pressures on the SMR from the maximum unstable block size can be obtained from non-linear FIA that includes the discontinuities or force equilibrium analyses.

The quality and amount of data retrieved from the site investigation and subsurface material testing programs dictates the levels of epistemic uncertainty and aleatory variability that needs to be accounted for in the SMR analysis and design. Geotechnical input parameters are developed for use as input for the analyses of the SMR structures that adequately account for:

- the epistemic uncertainties related to the lack of knowledge and biases in the determination of site-specific subsurface material properties including uncertainties related to the methods used for the development of geotechnical parameters from empirical relationships, and

- the aleatory variabilities related to the natural randomness of the subsurface properties including spatial variability and the precision of soil and rock measurements.

The epistemic uncertainties are addressed by using multiple geotechnical parameter predictions obtained from different measurement techniques and empirical relations along with associated weights reflecting the level of confidence placed on each prediction. Probabilistic distributions, usually normal or log-normal, are used to address the aleatory variability of the measured geotechnical parameters. Mean (μ_i) and standard deviation (σ_i) are calculated or assigned for each case of epistemic uncertainty (i) together with a weight factor (w_i) reflecting the confidence in the geotechnical parameter prediction. Weighted mean value (μ) and the weighted variances (Var) that include the site epistemic uncertainty of the estimated geotechnical parameter are calculated as follows:

$$\mu = \sum_i w_i \mu_i$$
$$\mathrm{Var} = \sum_i \left[w_i \sigma_i^2 + w_i (\mu_i - \mu)^2 \right] \tag{6}$$

The weighted average (μ) values are adopted as the best estimate (BE) of the geotechnical parameter. The calculated variations (Var) are used to define the lower bound (LB) and upper bound (UB) estimates of the geotechnical parameter.

The design of SMR structures must consider different environmental and plant operating loads that are combined in different design load combinations per the governing design codes. The superposition principle, which is applicable only for linear elastic analyses, is essential for the design because it allows the results of the different dynamic, static and thermal stress analyses to be combined in different load combinations. The linear elastic assumption also allows the analyses to be performed on structural models with many degrees of freedom and eliminates the need for defining initial conditions for each design load combination to calculate the structural design demands. Equivalent linear properties of soil and rock materials are developed from data collected from field and laboratory tests ~~that~~ to provide conservative design of deeply embedded SMR.

To address the uncertainties related to the variations and determination of soil and rock dynamic properties and site subgrade conditions, at least, three sets of seismic SSI analyses are performed using the BE, LB, and UB profiles of subgrade dynamic properties to address aleatory and epistemic uncertainties of subgrade conditions. Dynamic soil properties are used that are compatible to the strains generated in the subgrade by a design level earthquake event to address effects of primary nonlinearity of subgrade materials induced by the free field excitation. As shown in Figure 3, these hazard consistent strain-compatible properties are obtained from results of probabilistic SRA performed following the Approach 3 described in McGuire, at al. (2001).

The following subsurface properties are used to calculate the demands from static loads including earth pressure and thermal loads:

- effective unit weight that for soil materials below groundwater table represents the total weight of soil minus unit weight of water,
- elastic modulus representing linearized stiffness properties of the soil and rock at the site for long-term static loading conditions, and
- Poisson ratio representative of site-specific at rest lateral pressure conditions.

UB values for soil effective unit weight and Poisson ~~R~~ratio are used as input for the static SSI analyses to address uncertainties in the calculations of static earth pressure demands. LB estimates of subgrade elastic modulus are used for the static SSI analyses to obtain conservative estimates of subgrade deformations and structural stress demand under gravity and long-term static loading on the SMR structures. UB estimates of subgrade elastic modulus are used in the thermal analyses models to obtain conservative estimates of the thermal stress demands on the SMR structures.

As shown in Figure 3, earth pressure results from non-linear FIA and force equilibrium analyses can be used to demonstrate that the linear elastic static SSI design analyses provide conservative design demands that adequately address variations and uncertainties in the soil and rock properties including the effects of non-linear and anisotropic subsurface materials. To ensure there are sufficient margins in the earth pressure loads considered for the design, deterministic and probabilistic approaches described in NEDO-33914 (2022) may be implemented based on the non-linear analyses that include the effects of non-linear and anisotropic behaviour of subsurface materials.

The accuracy of the non-linear models to predict the soil and rock pressures, in general, may be increased by using more sophisticated models and increasing the number of model parameters. However, increasing the number of model parameters usually increases the uncertainty in the calculated soil pressure results. Therefore, simpler non-linear models with fewer input parameters are more typically used.

Non-linear constitutive models are used in the FIA to define the relationship between the stresses and strains in the soil and rock materials as well as subgrade-structure interfaces and rock discontinuities. The selection of these non-linear constitutive models for the FIA is based on the subsurface site characteristics and the expected stress levels that result from construction of the SMR. The Mohr Coulomb failure criterion is typically used to represent shear failure in soil. Soils with Mohr Coulomb behaviour experience purely linear, elastic deformation with increased stress until the stress is large enough to fail the soil and the behaviour turns fully plastic. The Mohr Coulomb model is adequate for most soil, rock and interface models although, in general, it is an oversimplification of the stress/strain behaviour of many materials under significant loading. The key inputs required for defining Mohr Coulomb constitutive models include the elastic modulus, Poisson’s ratio, dilation angle, friction angle and cohesion intercept. The Mohr Coulomb model is used for most of soil materials unless the anticipated soil behaviour cannot be modelled with a linear elastic behaviour and the use of a single elastic modulus.

The Mohr Coulomb failure criterion is also typically used to represent shear failure in the rock. The Mohr Coulomb failure criterion is sometimes also used to incorporate anisotropy with the ubiquitous joint model to include weak planes with specific orientations. The Generalized Hoek Brown (GHB) model may be used to better represent the non-linear stress strain behaviour of rock masses when Mohr Coulomb constitutive models are not considered representative over a larger range of stresses, specifically, the response of an isotropic rock masses where the rock stiffness is nearly constant over a range of stresses but the shear strength is variable due to the presence of discontinuities and weak zones (Hoek and Brown, 1997; 2019). The GHB model is applicable to rock masses with confining stresses below the transition to ductile failure, which is anticipated for most of candidate sites. The GHB criterion uses intact rock measurements of unconfined strength with the GSI geomechanical rock mass classification to estimate adjusted strength and deformation parameters, Mohr Coulomb friction angle and cohesion parameters, the uniaxial rock mass strength, and rock mass deformation modulus for the rock mass, under different geological conditions. The GHB failure criterion is established based on empirical equations such as the semi-quantitative one developed by Hoek et al. (2013).

CONCLUSSION

Generic geotechnical parameters are presented in this paper representing a wide range of subsurface conditions at most candidate sites for use as input for the standard design of deeply embedded SMRs. A set of eight generic profiles are developed using of database of measured properties at number of candidate sites representing the variation of dynamic and static properties of soil and rock materials with depth. The eight generic profiles represent a wide range of conditions varying from deep medium-stiff soil to shallow stiff soil/soft rock to hard rock sites.

Recommendations are provided for development of equivalent linear properties for soil and rock materials from data collected from field and laboratory tests to addresses variations and uncertainties in the subsurface conditions in the site-specific design of the deeply embedded SMR. Approaches for developing parameters for characterizing rock masses and modelling the subgrade non-linear and anisotropic behaviour are also provided for use as input for non-linear FIA. The results of these FIA and force equilibrium analyses can also be used to demonstrate that the design analyses provide conservative design demands with margins that adequately address variations and uncertainties in the subsurface conditions and the nonlinearity of the subgrade.

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