Sliding Evaluation and Results

Non-Proprietary Version

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ABSTRACT

The purpose of this Technical Report is to present the methodology and results of non-linear sliding analyses due to seismic events for the US-APWR Standard Plant.

This revision of the Report describes:

- Objectives and scope of the non-linear sliding time history analyses.
- General analysis method, including: Assumptions, Seismic loads, Material Properties, and Contact modeling.
- Specific aspects, including special features of the finite element models, and methodology for building, calibrating, and validating a computationally effective numerical model for non-linear sliding analyses.

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LIST OF ACRONYMS

The following list defines the acronyms used in this document.

A/B	Auxiliary Building
ASCE	American Society of Civil Engineers
CIS	Containment Internal Structure
DC	Design Certification
DCD	Design Control Document
DOF	Degrees of Freedom
EPRI	Electrical Power Research Institute
E/R	Electrical Room
FE	Finite Element
FoS	Factor of Safety
LMS	Lumped-Mass Stick
LMSM	Lumped-Mass Stick Model
NRC	U.S. Nuclear Regulatory Commission
PCCV	Prestressed Concrete Containment Vessel
PS/B	Power Source Building
R/B	Reactor Building
SRP	Standard Review Plan
SSE	Safe-Shutdown Earthquake
SSI	Soil-Structure Interaction
T/B	Turbine Building

LIST OF FIGURES

1.0 INTRODUCTION

Standard Review Plan (SRP) NUREG-0800 Section 3.8.5, Revision 2 (Reference 1) requires that Seismic Category I structures have a margin of safety for sliding expressed as a factor of safety (FoS) against sliding greater than 1.1. These FoS are typically calculated as the ratio of the minimum seismic resistance to the maximum seismic demand. Both resistance and demand are conservatively derived based on pseudo-static analyses and using conservative parameters that envelop a variety of site situations.

As stated in PVP2011-57600 (Reference 2), “in Design Certification (DC) applications it has been difficult to demonstrate seismic stability using simple calculations”. This is due to the nature of the DC applications, where the design parameters for both seismic demand and seismic resistance are not based on actual site investigations, but in general are conservatively specified representing envelopes of the site-specific characteristics to be expected at the potential sites (PVP2011-57600, Reference 2).

This issue is resolved by performing the seismic sliding evaluation for the Reactor Building (R/B) Complex and Turbine Building (T/B) structures of the US-APWR using a more realistic non-linear time history analysis. In addition to specifying a series of conservative assumptions and conservatively assessed parameters, a safety margin for seismic induced sliding is secured by amplifying all components of the input seismic motion by a factor of 1.1, as defined in the Civil Structure Design Criteria (Reference 3).

The resulting sliding displacements are multiplied by a FoS in accordance with the provisions of ASCE/SEI 43-05 (Reference 4) for the design of umbilicals, i.e., attached structures, pipe and/or equipment, and structure gaps.

2.0 PURPOSE AND SCOPE

The seismic sliding analysis is performed for the following Seismic Category I structures:

1. R/B Complex structures, including the R/B, Prestressed Concrete Containment Vessel (PCCV), Containment Internal Structure (CIS), Auxiliary Building (A/B), East Power Source Building (PS/B), and West PS/B.
2. T/B structures, including the T/B and the Electrical Room (E/R).

The structures are subjected to the Safe-Shutdown Earthquake (SSE). Five sets of acceleration time histories are used, as described in Section 4.2.3. The calculations are performed for six different soil profiles that are described in Technical Report MUAP-10006 Revision 3 (Reference 5). Both cracked and uncracked concrete properties are considered for the R/B Complex. Cracked concrete properties are considered for the T/B structures.

3.0 OBJECTIVE

The objective of this report is as follows:

1. Demonstrate acceptable seismic sliding of the R/B Complex and T/B structures: Calculate possible sliding during the SSE by means of non-linear time history analyses and demonstrate that the amount of sliding, if any, is too small to cause any physical damage to the R/B Complex and T/B structures or to the structural connections.
2. Obtain design values for umbilicals: Conservatively estimate upper bounds for seismic sliding to be used, along with other structural displacements, for the design of attached structures, pipe and/or equipment between buildings and other umbilicals as required.

4.0 METHODOLOGY

There are sixty analytical cases to evaluate for the R/B Complex and thirty cases for the T/B. They consist of five acceleration time histories for six soil cases for both cracked and uncracked structures. Thus: 5 x 6 x 2 = 60 (only the cracked section is analyzed for the T/B, hence 30 analysis cases). Since the lower modes of a structure control virtually all of the lateral force delivered to the foundation, these same modes control the sliding potential of the structure (Reference 6). Thus, the use of a detailed finite element (FE) model to compute sliding is not necessary nor is it computationally efficient. Consequently, Lumped-Mass Stick Models (LMSMs) that accurately reproduce the lower modes of the detailed structures are developed and used to evaluate the sliding potential of the R/B Complex and T/B.

The methodology used to evaluate sliding of the R/B Complex and the T/B is to develop Lumped-Mass Stick (LMS) Models of these structures using properties derived from the FE models, referred to herein as the dynamic FE models (see Section 4.3 for more detail). These dynamic FE models are the ANSYS models which are then translated to SASSI models that are used to compute the Soil Structure Interaction effects.

These dynamic FE models are modified herein to be able to compute sliding. The LMSMs are then refined using correlations to results obtained from dynamic FE model analyses. Once the LMSMs are developed to be suitable for sliding analysis, they will be used to compute sliding displacements for all the cases described above.

This Section describes the Assumptions (4.1) and Materials and Loads (4.2), followed by the modification of the dynamic FE models to enable them to accurately predict sliding displacements (4.3). Finally the methodology used to develop the LMSM is described (4.4).

4.1 Assumptions

1. Sliding is assumed to occur in some cases under the SSE.

4.2 Materials and Loads

4.2.1 Subgrade Properties

Subgrade properties, including soil layering, as well as stiffness and damping for each layer, are input parameters for the SSI analyses that provide the ground accelerations at the basemat-subgrade level, accounting for the effect of the soil profiles on seismic accelerations.

The most important subgrade parameter in sliding analyses is the coefficient of friction at the basemat-subgrade interface. This friction coefficient has different values for the case when the structure has not started sliding (static friction coefficient, μ_s) and for the case when the structure is already sliding (kinetic friction coefficient, μ_k).

Static friction coefficient, μ_s (rock subgrades):

The static friction coefficient characterizes the shear strength of the interface before any sliding occurs. Based on Reference 7, μ_s between mass concrete and various subgrade materials corresponds to a friction angle of 35° for rock and is equal to the friction angle at failure for soils.

The coefficient of friction between the basemat and rock is based on an angle of friction of 35° which is equivalent to a coefficient of friction of 0.7. The rock surface must be cleaned, and fissures and fractures filled in. The concrete-to-rock test results of Electrical Power Research Institute (EPRI) (1992) report (Reference 8) show angle of friction values that consistently exceed 35°.

The cold joint at the mud mat to bottom of foundation contact is a mechanical interface since the mud mat will not be smooth. It will be “raked” with a very rough surface (minimum amplitude greater than ¼ inch - as recommended in Reference 9, paragraph 11.7.9) to maintain a minimum friction coefficient of 0.7 for both dry and wet surfaces. This will be specified in the Design Control Document (DCD). Based on laboratory test results from over 150 specimens, Reference 8 indicates a lower bound of 57° for the peak friction angle at the concrete-to-concrete contact for cold joints.

Static friction coefficient, μ_s (granular soil subgrades):

Based on correlations in the literature, granular soils with properties corresponding to the requirements in MUAP-10006 (Reference 5), namely with shear wave velocities V_S > 270 m/s, have effective friction angles, Φ of approximately 40°. The governing friction occurs between the mud mat and the underlying granular soil where a thin soil layer exists that is interlocked with the bottom of the mud mat. Upon impending sliding conditions, this layer is constrained to move together with the mud mat. Therefore, sliding of mass concrete placed directly on granular soil is a shear failure phenomenon and the “friction coefficient” is an expression of the shear strength of the soil. A conservative value of Φ = 35°, corresponding to a friction coefficient μ_s = 0.7, is considered for this type of soil. In order to provide a suitable subgrade material, the soil should not include a significant amount of fines and be compacted to an adequate relative density (at least 65%, corresponding to dense sand) before the placement of the mud mat.

Static friction coefficient, μ_s (clay subgrades):

Any fine grained materials within a few feet below the basemat will be replaced by engineered fill. Engineered fill will be specified in the DCD as a well drained granular backfill with a minimum friction angle of Φ = 35°.

Based on the above considerations, a conservative value for the friction angle of natural soil materials below the basemat for all profiles is selected to be Φ = 35°, corresponding to a static friction coefficient μ_s = 0.7. This minimum angle of internal friction will be specified to be 35° in DCD Table 2.0-1 as a site requirement.

Kinetic friction coefficient, μ_k (all subgrades):

The kinetic friction coefficient characterizes the shear strength of the interface during sliding. This shear strength is also termed as “residual strength”. Based on results of a large number of laboratory and full scale tests (a comprehensive review is presented in Reference 10), the residual strength friction angle at concrete-to-concrete and concrete-to-rock interfaces exceeds 30°, corresponding to kinetic friction coefficients, $\mu_k > 0.57$.

The residual friction angle for granular soils is also known as the constant volume friction angle or the steady state friction angle (References 11 and 12). Been and Jefferies (Reference 12) report values of the residual friction angle based on a large number of laboratory soil tests with samples from seven different types of sand. Most values range between 30° and 32°, with extreme values of 27° and 35°, all yielding kinetic friction coefficients, $\mu_k > 0.5$.

Therefore, a conservative value of the friction coefficient during sliding, $\mu_k = 0.50$, is used in the nonlinear sliding analysis presented in this report for all subgrades.

Effect of Groundwater:

The friction coefficient between two surfaces in contact is in general affected by the presence of water, as the lubricating effect of water will usually reduce friction. However, the friction coefficient below the mud mat discussed here, representing friction between mass concrete and granular soil, is the result of shear failure in soil rather than of sliding between two surfaces. As discussed previously, this is because the cement penetrates in soil pores during mat placement and forces the failure surface to occur through soil rather than at the interface, and therefore the friction coefficient is an expression of the shear strength of soil. It was verified experimentally that the results in terms of effective stresses and the shear strength of granular soils were not affected by the presence of water in the soil sample (Reference 13). Therefore, it is considered in this analysis that presence of water does not affect the friction coefficient between mud mat and subgrade.

Regarding the concrete-to-concrete and rock-to-concrete interfaces, all experimentally based values of the friction coefficient (References 8 and 10) discussed previously were obtained for both dry and wet interfaces.

4.2.2 Structural Properties

The input parameters used for the FE models of the R/B Complex and T/B are the same as those used in the SSI analyses, and are discussed in detail in MUAP-10006 (Reference 5). One parameter that could not be replicated from the frequency domain analyses performed with ACS SASSI (Reference 14) to the transient analyses performed here with ANSYS (Reference 15) is the damping ratio. ACS SASSI uses a complex damping formulation, while in transient analyses performed with ANSYS, the damping ratio is described via Rayleigh damping, using the alpha-d and beta-d coefficients that multiply the mass matrix and the stiffness matrix, respectively. The viscous damping matrix is given by (Reference 15):

$$[C] = \alpha[M] + \beta[K] \qquad (4.1)$$

where M and K are the mass and stiffness matrices, respectively, and α and β are the Rayleigh damping matrix multipliers.

While the complex damping formulation allows use of a uniform damping ratio over the entire frequency range, Rayleigh damping models have a damping ratio that is approximately constant over a certain, predefined, frequency range, and has larger values outside that range. The usual approach is to bound the range of structural frequencies of interest (herein: from the lowest fundamental frequency in the horizontal directions to the frequency of the first vertical mode of vibration), and determine the values of α and β such that the desired damping ratio is matched to the frequencies bounding this range. In this manner, the damping ratios used in the analysis will be conservative since the modal damping in the frequency range of interest is less than or equal to the structural damping used in the SSI analyses.

Moreover, the complex damping formulation used in ACS SASSI allows assigning different damping values to different structural components, while the alpha-d coefficient used in Rayleigh damping applies to the entire model. The beta-d coefficient can still be input at the material level in transient analyses. More details regarding the definition of damping in transient analyses in ANSYS can be found in Reference 15.

In order to accommodate these differences in damping methods, damping is input in the FE models and the LMSM used in the non-linear analysis as follows:

1. Determine a range of modal frequencies relevant for sliding analysis for each sub-model (i.e. R/B+A/B+PS/Bs, PCCV, and CIS for the R/B Complex; and T/B and E/R for the T/B). The relevant frequencies for sliding analysis range between the lowest fundamental frequency in the horizontal direction and the fundamental frequency in the vertical direction.
2. Calculate one value of α for each model (R/B Complex or T/B) and several values of β for each sub-model, such that the appropriate damping ratios are input to each sub-model over its relevant frequency range. Use lower (conservative) values wherever an exact match cannot be achieved.

The target damping ratios for non-linear sliding analysis for the SSE are as follows (Regulatory Guide (RG) 1.61, Reference 16):

- Concrete elements: 4% for uncracked concrete and 7% for cracked concrete.
- Pre-stressed Concrete elements: 5%
- Welded and friction-bolted steel structures: 4%

4.2.3 Seismic Accelerations

The load acting on the structure and possibly inducing sliding is the ground acceleration. The input to the sliding analyses consists of net translation acceleration time histories extracted from the SASSI SSI analyses and applied to the subgrade below the basemat in all three directions. Five sets of acceleration time histories will be used in the non-linear sliding analyses for each of the six soil profiles, and for two types of structural models, i.e., cracked and uncracked. The five time histories are developed in compliance with the requirements of SRP 3.7.1 (Reference 17).

The subgrade is modeled as a rigid surface, and the effect of variability of ground motion between the different points of the basemat is neglected (Section 4.1, Assumption 0). To properly account for the effect of soil profile on ground accelerations arriving at the basemat level, the acceleration time histories at the basemat level are calculated in the SSI analysis. The ACS SASSI (Reference 14) computer program is used to perform the SSI analyses of both the R/B Complex and T/B FE models. The SSE ground motion is applied in three orthogonal directions simultaneously. The SSI analyses are performed for a set of six generic soil profiles. The acceleration time history responses obtained from these analyses is used as input for the non-linear sliding analyses presented herein. All components of the seismic input accelerations are amplified by a factor of 1.1, as explained in Section 1.0. To obtain values that are representative for the entire basemat area and to avoid any effects of local flexibility of the basemat, the horizontal and vertical accelerations computed in ACS SASSI are taken from a location below a major wall of the structure (R/B Complex or T/B) and close to the geometric center of the plan of the basemat.

This methodology, therefore, develops analyses that are fully coupled, i.e., [

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4.2.4 Backfill Pressures

4.3 Development of FE Models for R/B Complex and T/B

4.3.1 General

There are two FE models for the R/B Complex considered: one detailed FE model which is used to validate the dynamic model; and one dynamic FE model, which is used for dynamic analyses in ANSYS, and these two models have correlation with each other. The dynamic FE model is the basis for correlation with the LMSM. Contact (or sliding) elements and a rigid basemat are added to the dynamic FE model for non-linear sliding analyses. The dynamic FE model customized for sliding analyses is referred to herein as the FEM.

Since the T/B is a smaller and relatively simpler structure than the R/B Complex, there is only one FE Model for it.

4.3.2 Contact Formulation

As Revision 0 of this report is written before the actual start of relevant analyses for the R/B Complex and T/B, the analysis method described here will be illustrated with a numerical example using a model structure representative for the R/B Complex structures. The manner in which the dynamic FE model is developed into a non-linear time history analysis model is shown in Figure 4.3.2-1. [

]

[

] The final layout of the FEM prepared for sliding is shown in Figure 4.3.2-2.

The contact elements model Coulomb friction. They are also compression-only elements that are deactivated at locations and time instants where uplift occurs, therefore the uplift effects on sliding are captured by the numerical model.

The stiffness of the contact elements is selected so that spurious vibrations in both horizontal and vertical directions do not occur. These vibrations would be induced by rigid body modes of the entire structure (as a mass) placed on the contact elements (as springs characterized by their stiffness). [

] The expression, for harmonic input motion of circular frequency ω is (Reference 19):

$$TR = \sqrt{\frac{1 + [2\zeta(\omega / \omega_n)]^2}{[1 - (\omega / \omega_n)^2]^2 + [2\zeta(\omega / \omega_n)]^2}} \tag{4.2}$$

Where ζ is the damping ratio of the structure and ω_n is the natural frequency of vibration of the single degree of freedom (DOF) system represented by the entire mass of the structure placed on the elastic support,

$$\omega_n = \sqrt{\frac{k}{m}} \tag{4.3}$$

With k = stiffness of the support and m = mass of the entire structure, e.g. R/B Complex.

4.4 Development of Lumped-Mass Stick Model

4.4.1 General

The calibration and validation method for the LMSM used in non-linear sliding analyses are presented in Section 4.4.2. The purpose of the calibration (see Figure 4.4.1-1) is to design a LMSM that is able to accurately reproduce the amount of sliding calculated using the FEM. The LMSM developed for sliding consists of a rigid basemat with a stick model representing the structure. The stick model has concentrated masses at the levels of the main floors of the FEM, and beam elements in between that reproduce the story stiffness of the FEM levels. As for each main floor, all the structural and equipment masses are lumped into one single concentrated mass. This LMSM is validated only for sliding analysis, and is not appropriate for inferring any structural response other than seismic induced sliding.

The first phase in building the LMSM is deciding on the number and locations of lumped masses. As the largest part of the structural mass is generally located at the floor levels, lumped masses are located at the main floor elevations. Next, the walls and columns from each floor are assigned to the neighboring floor levels. This operation is illustrated in Figure 4.4.1-2, based on the analysis of an example structure with characteristics similar to those of the R/B Complex structures.

There are several models used in the non-linear sliding analyses, as follows:

1. The LMSM for sliding analysis of the R/B Complex consists of a rigid basemat of the same size and shape as the actual basemat of the R/B and three LMS sub-models, representing (1) R/B+A/B+PS/Bs, (2) PCCV, and (3) CIS. The three LMS sub-models act independently. In the LMSM representing the R/B Complex, these sub-models are attached to the common basemat at points corresponding to the location of their centers of mass in the horizontal plane. Two different LMSMs are developed for the R/B Complex: one corresponding to the uncracked section properties and the other to cracked section properties.
2. The LMSM for non-linear sliding analysis of the T/B consists of a rigid basement (including the actual basemat plus the wall and floors at the basement level) and two LMS sub-models, representing (1) T/B, and (2) E/R. The two LMS sub-models act independently. In the LMSM, these sub-models are attached to the common basement at points corresponding to the location of their centers of mass in the horizontal plane.

The calibration and validation methods discussed in the following Sections refer to the R/B Complex LMSM and the T/B LMSM (numbers 1 and 2 above).

4.4.2 Calibration Method

4.4.2.1 Basemat Modeling

The basemat of the R/B Complex is modeled with SHELL63 elements that are made into a rigid plate by means of the ANSYS command CERIG. The rigid plate is placed at the basemat level. The SHELL elements have mass equal to the mass of the basemat, and therefore the rotational inertia due to distributed mass is considered. The three LMS sub-models (R/B+A/B+PS/Bs, PCCV, and CIS) are rigidly connected to the rigid plate. They have masses placed at the elevation of main floors and are represented as lumped mass using MASS21 elements. The mass elements have translational mass MX = MY = MZ equal to the mass of each floor in the FEM, and rotational inertia about the vertical axis.

For the T/B, the basement is modeled by two parallel rigid plates made of SHELL63 elements. The two plates are connected to each other using solid elements, and are placed one at the basemat elevation, and the other at the top of the basement level. The mass of each plate is calculated in ANSYS from the dynamic FE model, to correctly reproduce all inertial properties of the basement. The two LMS sub-models (T/B and E/R) are rigidly attached to the upper plate.

4.4.2.2 Mass Analysis

Lumped masses are calculated from the dynamic FE model using the PSOLVE (Reference 15) command (partial solution) after selecting the structural elements associated with seismic mass at each level. Both the translational mass and the rotational mass about the vertical axis are calculated and applied to the LMSM. The center of mass location at each level is also obtained and used for determining the elevation of each lumped mass.

4.4.2.3 Stiffness Analysis

The stiffness of each story of the LMSM is calculated based on the corresponding story stiffness of the dynamic FE model. Due to much larger stiffness of the floors as compared to walls and columns, most lateral resistance is provided by the walls in shear. The shear stiffness of level "*k*" along DOF "*j*" is calculated with ANSYS using the conventional static method from the FEM. A unit displacement in direction "*j*" is applied at the top of level "*k*" along with restraining all DOF at the lower levels. All but one DOF (namely "*j*") are also restrained at level "*k*" and all the levels above (see Figure 4.4.2.3-1). The shear stiffness of level "*k*" along DOF "*j*", K_{jk}^{SH}, is provided by ANSYS as the sum of all forces along DOF "*j*" at the lower levels, F_{ji}.

$$K_{jk}^{SH} = \sum_{i=1}^{k-1} F_{ji} \tag{4.4}$$

To account for the additional flexibility of the structure due to axial deformations of walls and columns leading to some general bending of the structure (with the floor sloping), a "total stiffness" is approximately calculated at each floor level by applying a unit deformation as described above, but with allowance for rotation about the horizontal axis normal to the direction of unit displacement (see Figure 4.4.2.3-2). The total stiffness at each floor and in each horizontal direction, K_{jk}^{T}, is obtained with ANSYS in the same way as the shear stiffness in Equation 0. The LMSM is considered to have, at each floor, a shear stiffness equal to K_{jk}^{SH} in Equation 0, and a bending stiffness, calculated based on K_{jk}^{T} as follows:

$$K_{jk}^{B} = \left(\frac{1}{K_{jk}^{T}} - \frac{1}{K_{jk}^{SH}} \right)^{-1} \tag{4.5}$$

The stiffness in the vertical direction at each level is calculated with ANSYS similarly to the level shear stiffness, with unit displacements applied in the vertical direction. The rotational stiffness about the vertical axis at each level is calculated in the same manner, by applying unit rotations and summing up all moment reactions.

4.4.2.4 Modal Analysis

Modal analysis of the dynamic FE model is performed to derive the lower modes of vibration in each direction. Use of the first mode in each direction is supported by published evidence that the effect of higher modes on calculated sliding is negligible (References 6, 20 and 21). The natural frequencies and mode shapes of the LMSM are also calculated based on the structural characteristics determined as shown in Sections 4.4.2.2 and 4.4.2.3. Due to the structural characteristics of the R/B Complex and T/B structures, with mass mostly distributed at each floor level, as distributed floor mass or as concentrated equipment masses, a dominant vertical mode does not exist (see Figure 4.4.2.4-1 and Figure 4.4.2.4-2). Therefore, a first calibration of the LMSM is performed to match the first modes in the horizontal directions. This is done by adjusting the total story stiffness at each level, to match the fundamental frequencies and corresponding modal masses in the X and Y-directions.

Next, an equivalent fundamental frequency in the vertical direction of the dynamic FE model is estimated by considering a number of modes in the dynamic FE model such that the sum of modal masses equals the modal mass of the first mode in the vertical direction of the LMSM. The vertical stiffness at each level in the LMSM is adjusted to match the fundamental frequency estimated from the dynamic FE model. Since the matching of dynamic behavior in the vertical direction is important for the sliding response, fine tuning of fundamental vibration frequency in the vertical direction is performed using actual sliding analysis result comparison (Section 4.4.3.2).

4.4.3 Fine Tuning and Validation

4.4.3.1 Screening for Representative Cases

A few representative cases are selected from the 60 SSI analysis cases for the R/B Complex (and 30 analysis cases for the T/B) to be used to fine-tune and validate the LMSM based on the FEM sliding analysis results. This requires performing nonlinear sliding analyses with the FEM. The selection criteria are:

4.4.3.2 Fine Tuning of the LMSM

4.4.3.3 Validation Criteria

4.5 Conclusions of the Methodology

The following conservative assumptions are used in the non-linear sliding analyses:

1. Multiply ground acceleration by 1.1 in all three directions – this represents a margin of safety of 10% against pseudo-static sliding, but, based on results of sensitivity analyses, induces a significant increase in actual calculated sliding. (This data will be provided in Revision 1.)
2. Use a conservative value of the friction angle for backfill of 35° rather than the actual average value of 40°.
3. Use a conservative value for the friction coefficient between basemat and subgrade for all subgrade profiles based on the minimum value of the friction angle for natural soil of 35° (valid for the softer soil profiles, 270-200 and 270-500).
4. Use of the kinetic friction coefficient under static conditions (before initiation of sliding).
5. Neglecting passive soil reaction against embedded basement walls during sliding.
6. The maximum groundwater level (1 ft below grade), inducing large buoyancy forces, is assumed to occur at the same time as the SSE.

A conservative value of expected maximum sliding is obtained as *FoS* times calculated maximum sliding. ASCE/SEI 43-05 (Reference 4) recommends a factor of safety *FoS* = 2 to 3 be applied to resulting sliding, and specifies that *FoS* = 2 is appropriate for conservative analyses. Given the conservative assumptions listed above, and especially use of 110% of the input ground acceleration in all directions leading to significant increase in sliding, a factor of safety *FoS* = 2 is considered appropriate for the results of the sliding analyses presented here. This value will be specified in the DCD to be used for the design of umbilicals.

4.5.1 Sensitivity Analyses

Sensitivity analyses will be performed for the following parameters:

- Amount of Live Load considered in the sliding analysis
- Effect of groundwater level

- Effect of the value of friction coefficient used in sliding analysis
- Effect of the factor of 1.1 multiplying all three input acceleration components

Figure 4.3.2-1 Preparing the Dynamic FE Model for Sliding Analysis (example structure)

Figure 4.3.2-2 FEM Prepared for Sliding Analysis (example structure)

Figure 4.4.1-1 Methodology for LMSM Calibration

Figure 4.4.1-2 Mass Analysis for LMSM Calibration

Figure 4.4.2.3-1 Stiffness Analysis for Estimating Shear Stiffness in the LMSM

Figure 4.4.2.3-2 Stiffness Analysis for Estimating Total and Bending Stiffness in the LMSM

Figure 4.4.2.4-1 Modal Analysis of the Dynamic FE Model – Horizontal Modes of Vibration

Figure 4.4.2.4-2 Modal Analysis of the Dynamic FE Model – Vertical Modes of Vibration

5.0 CALCULATIONS AND RESULTS

To be provided in Revision 1 of this Report.

6.0 CONCLUSIONS

To be provided in Revision 1 of this Report.

7.0 REFERENCES

1. Foundations, NUREG-0800 Section 3.8.5, Revision 2, U.S. Nuclear Regulatory Commission, Washington, DC, March, 2007.
2. Structural Design Challenges in Design Certification Applications for New Reactors, PVP2011-57600, Proceedings of the ASME 2011 Pressure Vessels & Piping Division Conference, Miranda, M., Braverman, J., Wey, X., Hofmayer, H., and Xu, J., Baltimore, MD, July, 2011.
3. US-APWR Standard Design Civil Structure Design Criteria, N0-CF00003, Revision 3, Mitsubishi Heavy Industries, Ltd., December, 2010.
4. Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, ASCE/SEI 43-05, American Society of Civil Engineers, 2005.
5. Soil Structure Interaction Analyses and Results for the US-APWR Standard Plant, MUAP-10006, Revision 3, Mitsubishi Heavy Industries, Ltd., July, 2012.
6. Simplified Procedures for Estimating Earthquake-Induced Deviatoric Slope Displacements, Journal of Geotechnical and Geoenvironmental Engineering, ASCE 133(4): 381-392, Bray. J.D. and Travasarou, T., 2007.
7. Foundation Analysis and Design, 5th Edition, Bowles, J.E., Mcgraw-Hill, 1996.
8. Uplift Pressures, Shear Strengths, and Tensile Strengths for Stability Analysis of Concrete Gravity Dams, Volume 1, prepared by Stone and Webster Engineering Corporation for Electric Power Research Institute (EPRI), Denver, Colorado, August, 1992.
9. Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary, ACI 349-06, American Concrete Institute, September, 2007.
10. Working Group on Sliding Safety of Existing Gravity Dams, Final Report, ICOLD European Club, G. Ruggieri (coordinator), 2004.
11. The Stress Dilatancy Relation for Static Equilibrium of an Assembly of Particles in Contact, Proc. Royal Society of London, Series A, pp. 500-527, P.W. Rowe, 1962.
12. A State Parameters for Sands, Geotechnique, 35(2):99-107, Been. K and M.G. Jefferies, 1985.
13. Constant Volume Cyclic Simple Shear Testing, Proceedings of the 2nd International Conference on Microzonation, San Francisco, CA, Finn, W.D.L., Laid, Y.P. and Bhatia, S.K.,pg 839-851, 1978.
14. ACS SASSI, Version 2.3.0 including “Option A” & NQA “Option FS”, “An Advanced Computational Software for 3-D Dynamic Analysis Including Soil Structure Interaction,” User Manuals Revision 5.0, Ghiocel Predictive Technologies, Inc., January 31, 2012.
15. ANSYS, Release 13.0, SAS IP, Inc., 2010.
16. Damping Values for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.61 Revision 1, U.S. Nuclear Regulatory Commission, Washington, DC, March 2007.
17. Seismic Design Parameters, NUREG-0800 Section 3.7.1, Revision 3, U.S. Nuclear Regulatory Commission, Washington, DC, March 2007.
18. Seismic Analysis of Safety-Related Nuclear Structures and Commentary, ASCE 4-98, American Society of Civil Engineers, 1998.

19. Dynamics of Structures, 2nd Edition, Clough, R.W. and Penzien, J., Mcgraw-Hill, 1993.

20. Simplified Seismic Slope Displacement Procedures in Earthquake Geotechnical Engineering, pp 327-353, Bray J.D. and K.D. Pitilakis (ed.), Springer, 2007.

21. Nonlinear Coupled Seismic Sliding Analysis of Earth Structures, Journal of Geotechnical and Geoenvironmental Engineering, ASCE 126(11):1002-1014, Rathje, E.M. and Bray, J.D., 2000.