January 2, 2013

Mr. Mark McBurnett, Senior Vice President
Oversight & Regulatory Affairs
Nuclear Innovation North America, LLC
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Lake Jackson, TX 77566

SUBJECT: REGULATORY AUDIT SUMMARY OF SOUTH TEXAS PROJECT UNITS 3 AND 4 COMBINED LICENSE APPLICATION - MAY 23 THROUGH MAY 27, 2011, STRUCTURAL ANALYSES

Dear Mr. McBurnett:

By letter dated September 20, 2007, South Texas Project Nuclear Operating Company submitted to the U.S. Nuclear Regulatory Commission (NRC) a combined license (COL) application to construct and operate two reactor units (Units 3 and 4) based on the U.S. Advanced Boiling-Water Reactor (ABWR) Design Certification at the South Texas Project Nuclear Power Plant.

On January 24, 2011, Nuclear Innovation North America (NINA) became the primary applicant for the license for these two units. The NRC Office of New Reactors (NRO) is reviewing the South Texas Project (STP) Units 3 and 4 COL application that incorporates by reference the ABWR Design Control Document. As part of this review, the NRO Structural Engineering Branch 2 conducted an audit of the documentation supporting Chapters 3.7 and 3.8 request for additional information responses and the radwaste building design of the STP COL application. The audit was conducted at the Sargent & Lundy office in Chicago, Illinois, from May 23 through May 27, 2011. The NRC staff followed the guidance in NRO Office Instruction NRO-REG-108, “Regulatory Audits,” in performing this audit. Enclosure 1 is a list of the NRC staff and NINA team participating in the audit. Enclosure 2 is the detailed results of the audit. Enclosure 3 is an action item list from the audit.

Please contact Tom Tai at (301) 415-8484 or Tom.Tai@nrc.gov if you have any questions related to the audit.

Sincerely,

/RA/

Ronaldo Jenkins, Branch Chief
LB3 Projects Branch
Division of New Reactor Licensing
Office of New Reactors

Docket Nos.: 52-012
52-013

Enclosures:
1) List of participant
2) Detailed Audit Result
3) Punch List for STP

cc w/encls: See next page

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MAY 23 THROUGH MAY 27, 2011

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DETAILED AUDIT RESULTS FOR SEISMIC ANALYSES
MAY 23 THROUGH MAY 27, 2011

1. Introduction

By letter dated September 20, 2007, STP Nuclear Operating Company submitted to the U.S. Nuclear Regulatory Commission (NRC) a combined license (COL) application to construct and operate two reactor units (Units 3 and 4) based on the U.S. Advanced Boiling-Water Reactor (ABWR) Design Certification at the South Texas Project Nuclear Power Plant. On January 24, 2011, Nuclear Innovation North America (NINA) became the primary applicant for the license for these two units. The NRC Office of New Reactors (NRO) is reviewing the South Texas Project (STP) Units 3 and 4 COL application that incorporates by reference the ABWR Design Control Document (DCD). As part of this review, the NRO Structural Engineering Branch 2 conducted an audit of the documentation supporting Chapters 3.7 and 3.8 request for additional information (RAI) responses and the technical issues associated with the computer code, SASSI, identified by the Defense Nuclear Facilities Safety Board's (DNFSB). The audit was conducted at the Sargent & Lundy (S&L) office in Chicago, from May 23 through May 27, 2011.

Representatives from the NRC, key technical personnel representing STP Units 3 and 4, NINA, Toshiba America Nuclear Energy, S&L, and NINA consultants were present during the audit.

The NRC staff followed the guidance in NRO Office Instruction NRO-REG-108, "Regulatory Audits," in performing this audit.

2. Objectives and Approach

The purpose of this audit is to review the structural design calculations and supporting documents in various RAI responses and the resolution of SASSI in Chapters 3.7 and 3.8. In addition, technical issues identified during the March 14 through March 18, 2011 audit (ML111260469) will also be discussed.

The scope of this audit includes Category I structures (Ultimate Heat Sink and Reactor Service Water Pumphouse and Tunnel (UHS/RSWPH and RSWT), Diesel Generator Fuel Oil Storage Vault (DGFOSV), and Diesel Generator Fuel Oil Tunnel (DGFOT)), non-Category I structures that are designed to meet seismic II/I requirements (Radwaste Building (RWB), Service Building (SB), Turbine Building (TB), and Control Building Annex (CBA)). It is the intent of both the applicant and staff that all open issues associated with Chapters 3.7 and 3.8 will be clarified and resolved to finalize the safety evaluation reports.

3. Technical Review

The following is a list of analyses reviewed during the audit for both Chapters 3.7 and 3.8:

1. Calculation U7-SITE-C-CALC-DESN-6019, Revision B, “Soil Pressure Profiles Between Reactor Building, RSW Tunnel, and Radwaste Building (Licensing)”
2. Calculation U7-SITE-C-CALC-DESN-6020, Revision B, “Soil Pressure Profiles Between Reactor Building, DGFOSVs and UHS – Pump House Building (Licensing)”
3. Calculation U7-YARD-S-CALC-DESN-6003, Revision A, “Diesel Generator Fuel Oil Storage Vault Tunnels Stability Evaluation”
4. Calculation U7-YARD-S-CALC-DESN-6006, Revision B, “Basic Structural Design of Diesel Generator Fuel Oil Tunnels (DGFOT)”
5. Calculation U7-RWB-S-CALC-DESN-6006, Revision A, “Radwaste Building Seismic Analysis (Licensing)”
6. Calculation No. 25425-000-KOC-0000-00017, Revision 0, “Effect of Groundwater Level Change on Dynamic Engineering Properties”
7. Calculation U3-SB-S-CALC-DESN-2100, Revision C, “Calculation of Service Building Stability Factors of Safety”
8. Calculation U3-TB-S-CALC-DESN-2100, Revision B, “Calculation of Turbine Building Stability Factors of Safety”
9. Calculation U7-CBA-C-CALC-DESN-6001, Revision B, “Control Building Annex Stability Evaluation (Licensing)”
10. Calculation U7-UHS-S-CALC-DESN-6002, Revision C, dated March 11, 2011, “Structural Evaluation of the Ultimate Heat Sink and RSW Pump House”
11. Calculation U7-UHS-C-CALC-DESN-6001, “Soil-Structure Interaction Analyses of Ultimate Heat Sink - Pump House Buildings, STP 3 & 4”
12. Calculation U7-UHS-C-CALC-DESN-6003, Revision B, dated January 6, 2011, “Stability Check of Ultimate Heat Sink and RSW Pump House”
13. Calculation U7-UHS-C-CALC-DESN-6007, “Soil-Structure Interaction Analyses of UHS Pump House Buildings, STP 3 & 4 for Empty Basin”
14. Calculation U7-RWB-S-CALC-DESN-6003, Revision A, dated January 6, 2011, “Radwaste Building Stability Evaluations (Licensing)”
15. Calculation 25425-000-HOC-HXYN-00005 (7 pages –Hurricane wind speed parameters) - prepared by Bechtel, approved 2/12/2012
16. Calculation U7-STE-S-CALC-DESN-6004, Revision A, “Evaluation for Design Basis Hurricane Loads”
17. Calculation U7-RWB-S-CALC-DESN-6005, Revision C, dated November 10, 2010, “Radwaste Building Structural Evaluation”
18. Calculation U7-RWB-S-GDD-6001, “Structural Design Criteria for Radwaste Building”

19. Calculation U7-RWB-S-CALC-DESN-6003, Revision A, dated April 26, 2010, “Radwaste Building Stability Evaluations (Licensing)”.
20. Calculation U7-YARD-C-CALC-DESN-6001, Revision B, “Diesel Generator Fuel Oil Storage Vault SSI Analysis”
21. Calculation U7-YARD-C-CALC-DESN-6002, Revision B, “Diesel Generator Fuel Oil Storage Vault and Tunnels SSI Analysis”
22. Calculation U7-RSW-S-CALC-DESN-6001, Revision D, “Basic Structural Design of Reactor Service Water Tunnel”

3A Summary technical issues reviewed under the scope of Chapter 3.7:

Impact of Subtraction Method on SSI Analyses of STP 3 & 4 Site-Specific Structures

STP has used the Subtraction Method (SM) in SASSI2000 for seismic SSI analysis of site-specific structures such as the UHS/RSW Pump House and RSW Piping Tunnel. NRC discussed with NINA the DNFSB recent findings regarding the technical adequacy and proper validation of the SM in SASSI2000. Following extensive discussions on this issue, NINA made a technical presentation on the subject to address the use of the SM in the STP 3C& 4 project.

NINA’s presentation, “Impact of Subtraction Method on STP Project, 5-24-11,” compared selected results from 3-D SSI analyses of the Control Building (CB); 2-D SSSI analyses of the Reactor Building (RB), RSW Piping Tunnel and RWB; and 3-D SSI analyses of the UHS/RSW Pump House with UB soil case – using both the Subtraction and Modified Subtraction Methods (MSM). Based on the results, NINA concluded that the two methods were comparable in terms of computed In-Structure Response Spectra (ISRS), and structural forces and dynamic soil pressures on walls. NINA concluded that the DNFSB’s concerns are thus unnecessary: SM does not adversely impact the STP 3 & 4 SSI analysis results. To further evaluate the use of the SM, NRC also requested comparisons of the results at additional locations in the UHS/RSW Pump House model -- locations considered more critical to structural response. NINA agreed to generate these additional results, which were then discussed later.

Assessment of the Effect of Change in Water Table from Elevation 25.5 ft to El. 28 ft on GMRS and FIRS for the STP Site, Bechtel, Licensing Document , April 18, 2011

This document, which addresses the impact of ground water variation on ground motion response spectra (GMRS) and foundation input response spectra (FIRS), is related to ACRS Question #58. NRC reviewed this document and agreed with its conclusion that a rise in the ground water level from an elevation of 25.5 feet to 28.0 feet has minimal effect on GMRS and FIRS. A 2.5-foot rise in the ground water level results in a decreased confining pressure of about 156 pounds per square foot (psf), which is small compared to the range of testing pressures (3,600-14,400 psf) needed to develop soil strain-dependent shear modulus and damping ratio relationships used in ground response analysis. In addition, the effect on low-strain shear wave velocity is also small, rendering the effect on GMRS and FIRS calculation negligible.

To evaluate the impact of a 2.5-foot increase in ground water level on SSI analysis results, NINA performed a sensitivity analysis of the DGFOSV in which the Poisson's ratio of soil layers was adjusted to reflect the rise in ground water. The analysis was performed for bounding in-situ lower bound (LB) and backfill upper bound (UB) soil cases. The results showed a small increase in the calculated ISRS at frequencies above 8 Hz due to the effect of the 2.5-foot rise in the ground water level. It was indicated that this small increase will be addressed by NINA at the design level.

Results of Poisson's Ratio Confirmatory Analysis

NRC shared with NINA the results of a confirmatory SSI analysis of the UHS/RSW Pump House using high Poisson's ratios for saturated soil layers with a cap of 0.495. The results in terms of 5 percent damped raw spectra calculated at several key locations in the structure for the Empty Basin-UB soil, Empty Basin-LB soil and Full Basin-LB soil cases were found to be enveloped by the NINA Design Spectra. NRC used the MSM for this confirmatory SSI analysis.

Validation of SASSI2000 Subtraction Method

To further address the issues raised by the DNFSB, NINA also validated the use of the SM in SASSI2000. The validation problem showed significant departure of the calculated transfer functions from the target solutions using the SM. To address this departure, NINA will add a cautionary note regarding this in the SASSI2000 release memos and SVVR (Validation Package), as follows:

"Issues have been identified with the use of the Subtraction Method as documented in PIP No. 2011-0539. In addition, special consideration should be given to cases with $a_o > 3$ (a_o is a non-dimensional equivalent foundation frequency, $a_o = 2\pi fr / V_s$, with f being the frequency of analysis, r the radius, and V_s the shear wave velocity of the soil." The use of the Subtraction Method shall therefore be approved by the Process Owner for Earthquake Engineering and the Project Manager."

The staff indicated that the memo would then be acceptable.

New SSI Results of UHS/RSW Pump House Seismic analyses using SM

NINA presented new results of the UHS/RSW Pump House SSI analysis (original SSI model) with UB soil case. The results, which include comparisons of transfer functions and 5 percent damped ISRS in the x-, y- and z-directions, were generated using SM and MSM. For certain locations in the structure, the MSM produces somewhat higher computed ISRS when compared to those previously obtained via SM. For the Pump House Roof, the design spectral accelerations are exceeded for f > 12 Hz in the vertical direction, and for the Mid-level of Basin Walls, the design spectral accelerations are significantly exceeded for 10 Hz < f < 14 Hz. Because MSM is considered to be more accurate than SM, NINA will scale up the current design spectra using the results from MSM to cover the above exceedances.

Calculation U7-SITE-C-CALC-DESN-6019, Revision B, "Soil Pressure Profiles Between Reactor Building, RSW Tunnel, and Radwaste Building (Licensing)"

This calculation contains the SSSI analyses and results for the RB, RSW Piping Tunnel and RWB. NRC reviewed the results for 3 documented soil cases: UB in-situ soil, UB backfill soil, and LB in-situ soil. 2-D SSSI analyses were performed using SM in SASSI2000. In general, the calculated SSSI pressures with UB backfill soil do not differ more than 10 percent from those obtained with UB in-situ soil except at a few locations (e.g., the east and west RSW Tunnel walls at elevations of 25.75 to 22.25 feet differ by 32.5% and 18.5%, respectively). In most cases the UB backfill soil results in higher calculated wall pressures. NINA will address the impact of SM on the calculated pressures.

Calculation U7-SITE-C-CALC-DESN-6020, Revision B, "Soil Pressure Profiles Between Reactor Building, DGFOSVs and UHS – Pump House Building (Licensing)"

This calculation contains the SSSI analyses and results for the UHS/RSW Pump House, DGFOSVs and RB combined model with UB backfill soil ranging from ground surface to the bottom of the RB mud mat for both in-fill and far field soils. Below the bottom of the RB mud mat, UB in-situ soil properties are used. An additional analysis was performed with LB in-situ soil for the entire combined model. 2-D SSSI analyses were performed using the SM in SASSI2000. For the combined model with UB backfill soil, the calculated pressures were generally higher than those for the combined model with LB in-situ soil. Because the model uses a soil column attached to the basement walls to output stresses from soil elements in order to calculate dynamic soil pressures on walls, NINA was asked to validate the stress output for 2-D soil elements in SASSI2000. NINA will address the impact of SM on the calculated pressures.

RAI 03.07.01-27, Supplement 3 Response

NINA provides background information on the calculated dynamic soil pressures on different building walls from 2-D SSSI analyses. These buildings and results include:

Control Building	Figure 3A-302
Reactor Building	Figures 3A-301 and 3H.1-1 through 3H.1-6
Radwaste Building	Figures 3H.3-350 and 3H.3.351
RSW Piping Tunnel	Figures 3H.6-212 through 3H.6-217
UHS/RSW Pump House	Figures 3H.6-218 through 3H.6-220
DGFOSV	Figures 3H.6-226 through 3H.6-231
DGFOT[1]	Figures 3H.7-5 through 3H.7-8

In Figure 3H.1-2, the calculated soil pressures for the RB West Wall that includes the effect of the adjacent RSW Tunnel and RWB exceed the DCD pressures at depths of 40-45 feet and 51-59 feet below ground surface. NINA was asked to address these exceedances.

In Figure 3H.6-219, the calculated soil pressures for the RSW Pump House North Wall at depths of 13-43 feet below ground surface are zero. NINA explained that the zero soil pressure is due to an air gap between the Pump House structure and adjacent building,

[1] Diesel Generator Fuel Oil Tunnel.

resulting in zero lateral soil pressure. This FSAR figure also demonstrates that a comparison of pressure plots for the RSW Pump House north wall shows the design pressure envelope representing total lateral soil pressures (i.e., static and seismic) while the other pressure profiles represent the seismic component only. NINA was asked to modify the design soil pressure to represent the seismic pressure only to maintain consistency with the other pressure profiles. In addition, there is a large increase in lateral soil pressures for the RSW Piping Tunnel at depths of 8-11 feet below ground surface (as shown in FSAR Figure 3H.6-213). NINA explained that this shift of the pressure spike from near the surface to these depths is due to the softer nature of the back soil (LB in-situ soil case) and the presence of an internal slab of the RSW Tunnel at this elevation, which attracts larger soil reaction forces. NINA also provided a new figure for review showing the pressure distribution profile for the UB backfill soil case.

In Figure 3H.6-229, the calculated soil pressures for the DGFOSV enveloped for all soil cases exceed the design pressures at depths of 7-11 feet below ground surface. NINA was asked to address these exceedances.

In Figure 3H.7-5, the calculated soil pressures for the DGFOT east wall enveloped for all soil cases (1,680 psf) slightly exceed the design pressures (1,600 psf) for depths greater than 9 feet below ground surface.

Calculation U7-RWB-S-CALC-DESN-6006, Revision A, "Radwaste Building Seismic Analysis (Licensing)"

This calculation details the development of the RWB structural model and analyses using the response spectrum method to calculate forces for stability evaluation. The response spectrum analyses were performed using the S&L in-house program DYNAS. Input for the program was obtained from Calculation U7-RB-C-CALC-DESN-6021, which was reviewed (without attachments). Figure A-8.319 shows the broadened spectral envelope for all soil cases and Regulatory Guide (RG) 1.60 anchored to 0.13g. This spectrum was used as input to the RWB model. The DYNAS analyses used 4 percent damping. The stability analyses and results are documented in Calculation U7-RWB-S-CALC-DESN-6003. There were no questions regarding this calculation.

Calculation No. 25425-000-KOC-0000-00017, Revision 0, "Effect of Groundwater Level Change on Dynamic Engineering Properties"

This calculation addresses the impact of ground water level variation on GMRS and FIRS. The groundwater level used to develop the soil properties assumes a water table elevation of 25.5 feet at the STP 3 & 4 sites. But the design water level is set at an elevation of 28 feet. This document evaluates the effect of this difference on engineering properties used to develop GMRS and FIRS. Calculations in the document support the fact that although an increase in ground water level results in a reduction of effective overburden stress, the change is very small and has negligible effect on dynamic soil properties (i.e., unit weight, shear wave velocity, strain-dependent soil shear moduli and damping ratios, and Poisson's ratio). There were no questions regarding this document.

Determination of Seismic Demand for Non-Seismic II/I Structures for Stability Evaluations

NINA provides information describing the way in which input motion is determined for the RWB and CBA for II/I stability evaluations, taking into account the impact of nearby heavy buildings (RB in the case of RWB, and CB in the case of CBA). The procedure for developing input spectra for RWB and CBA seismic II/I evaluations is described as follows:

- Add 5 nodes to the SSI model of the RB and CB at RWB and CBA locations at grade and foundation levels.
- Calculate 7 percent damped response spectra for the above nodes from SSI analyses.
- Calculate the average spectra for 5 nodes at grade and foundation levels.
- Envelope the spectra at the two levels.

No FIRS were generated for the RWB and CBA because they are non-Category I structures.

Seismic stability of the CBA structure was evaluated using the pseudo-static analysis procedure. The surface-supported CBA was modeled as a single-degree-of-freedom system. For horizontal direction, the horizontal ZPA was applied to the base and peak of the input horizontal spectra was applied to the mass point. For vertical direction, the vertical ZPA was applied to the base and 1.5 times peak of the input vertical spectra was applied to the mass point. The larger factor in the vertical direction accounts for the out-of-plane response of the roof. This calculation was found to be acceptable.

NINA provided the following clarifications regarding the way in which input motions were developed for II/I stability and design of site-specific non-Category I structures.

<u>II/I Stability Evaluation</u>: The RWB and CBA use enveloped site-specific SSE spectra and amplified site-specific SSE spectra. Not required for the TB and SB.

<u>II/I Design</u>: The RWB and CBA use enveloped site-specific SSE spectra and 0.3g RG 1.60 spectra. The TB and SB use 0.3g RG 1.60 spectra as input.

NINA was asked to explain why the SB does not use enveloped site-specific SSE spectra and 0.3g RG 1.60 spectra like the RWB and CBA. NINA agreed to change the criteria for the SB to maintain consistency with the RWB and CBA.

There were no other questions regarding this issue.

U3-SB-S-CALC-DESN-2100, Revision B, "Calculation of Service Building Stability Factors of Safety"

This calculation contains the seismic stability analysis for the non-Category I SB. The SB is designed such that damage to safety-related functions does not occur under seismic loads

corresponding to the SSE design motions. Therefore, the stability criteria used in determining margins of safety is similar to that for seismic Category I structures.

Input motion to the SB for II/I stability analysis consists of the amplified ground motion from SSSI analysis documented in Calculation U7-RB-C-Calc-DESN-6021.

Dynamic response of the SB was analyzed using the response spectra method via the RISA-3D program. The structure was modeled as a stick model on fixed base. Total base shear was calculated by adding the base shear from the stick model to the base shear from the basemat. Several specifics were not included in the calculation document, and NINA was asked to clarify certain details from the calculation originator (Fluor) – details such as 1) how modes are combined and 2) how earthquake directions are combined. Approval of this calculation depends upon a review of these details.

U3-TB-S-CALC-DESN-2100, Revision B, "Calculation of Turbine Building Stability Factors of Safety"

This calculation contains the seismic stability analysis of the non-Category I TB. The TB is designed such that damage to safety-related functions does not occur under seismic loads corresponding to SSE design motions. Therefore, the stability criteria used in determining margins of safety is similar to that for Category I structures.

Input motion to the TB for II/I stability analysis consists of the site-specific SSE design motion (i.e., modified RG 1.6 anchored to 0.13g). Dynamic response of the TB was analyzed using the response spectra method via the RISA-3D program. The structure was modeled as a stick model on fixed base. Separate response spectra analyses were performed for the north-south, east-west and vertical directions. The results of the response spectra analysis (RSA) were input to a static model of the TB to determine base shear and overturning moments for stability evaluations. The distance above the base elevation, where the horizontal component of seismic forces is applied, was determined by dividing the calculated overturning moment by the corresponding base shear.

NINA was asked to provide the calculation document, which details the model development and dynamic analyses of the TB to develop dynamic forces for stability evaluations. The subject document did not contain any significant details in this regard. Fluor, which performed the dynamic analysis of the TB, will provide this calculation for NRC's review. Approval of this calculation depends upon a review of these details.

U7-CBA-C-CALC-DESN-6001, Revision B, "Control Building Annex Stability Evaluation (Licensing)"

This calculation contains the seismic stability analysis of the non-Category I CBA. The CBA is designed such that damage to safety-related functions does not occur under seismic loads corresponding to SSE design motions. Therefore, the stability criteria used in determining margins of safety is similar to that for Category I structures.

Input motion to the SB for II/I stability analysis consists of the amplified ground motion from SSSI analysis documented in Calculation U7-SITE-C-Calc-DESN-6015.

Dynamic response of the SB was calculated using the pseudo-static method. The model is a single-degree-of-freedom stick with static load equal to 1g in the horizontal direction and 1.5g in the vertical direction. Resultant response from three global earthquake directions was obtained via the 100-40-40 rule. The basemat was accelerated at zero peak acceleration (ZPA), and forces were added to the forces obtained from the pseudo static analysis to determine the total base shear. Damping values of 4 percent for concrete and 3 percent for steel were used, and spectrum values were interpolated from 2 percent and 5 percent spectra.

Technical Presentation: Analysis of UHS Basin Columns

NINA presented the analysis methodology and results for the UHS Basin Columns. In SSI analysis of the UHS, hydrodynamic mass of the submerged parts of the columns were not included. To address its effect, NINA performed an uncoupled analysis of a single column with several boundary conditions at the top covering different existing conditions. The base and top of the columns were excited with response spectra envelopment from all cases used in the analysis of the UHS. Accelerations along the height of the column were calculated and the results will be used to design the columns. Note that the hydrodynamic mass of these columns is small compared to the hydrodynamic mass of the entire basin. The effect on global response of the UHS is considered insignificant. In addition, there is a large difference between the lateral soil pressures calculated from 2-D SSI analysis of the RSW Pump House and those calculated from 3-D SSI analysis of the RSW Pump House with full and empty basins (as shown in FSAR Figure 3H.6-219). STP informed that the lower pressures calculated from 2-D SSI analysis are not used for design. STP was asked to validate soil pressures calculated using soil element stresses from 2-D SSI analysis if such stresses will be used for design.

U7-UHS-C-CALC-DESN-6001, Revision B, "Soil-Structure Interaction Analyses of UHS Pump House Buildings, STP Units 3 & 4", and U7-UHS-C-CALC-DESN-6007, Revision A, "Soil-Structure Interaction Analysis of UHS Pump House Buildings, STP Units 3 & 4 for Empty Basin"

Table 3 in Calculation U7-UHS-C-CALC-DESN-6007, Revision A for empty basin lists the fixed-base frequencies of the full basin model. Although the title of the table is correct, it was confusing since NINA did not calculate fixed-base frequencies of the empty basin and, for that reason, they were not reported in this calculation (the full basin frequencies are shown instead). The empty basin model was developed from the full basin model by simply removing hydrodynamic added mass. NINA agreed to delete this table from Calculation U7-UHS-C-CALC-DESN-6007.

Validation of Element Forces and Stresses in SASSI2000

NRC reviewed three validation and verification (V&V) test problems to validate the accuracy of forces and stresses calculated in SGH-SASSI2000. This validation is necessary because forces from the spring element and stresses from the 3-D solid and 2-D plane-strain elements are used to calculate lateral soil pressures for design.

1) Verification Problem # SAS-5A, Revision 3, "Verification of SASSI2000 – Force Calculations in 3-D Spring Elements," SGS V&V, December 3, 2010.

The purpose of this problem is to verify SASSI2000's calculation of forces in 3-D spring elements.

2) Verification Problem # SAS-5E, Revision 3, "Verification of SASSI2000 – Stress Calculations in Solid Elements," SGS V&V, December 17, 2010.

   The purpose of this problem is to verify SASSI2000's calculation of stresses in 3-D solid brick elements.

3) Verification Problem # SAS-5H, Revision 1, "Verification of SASSI2000 – In-Plane Stress Calculations in 2-D Plane Stress Elements," SGS V&V, December 17, 2010.

   The purpose of this problem is to verify SASSI2000's calculation of in-plane stresses in 2-D plane stress elements.

All three verification problems show good agreement with the target solutions.

3B Summary technical issues reviewed under the scope of Chapter 3.8:

The staff conducted a detailed review of selected portions of the calculations listed in Section 3, "Technical Review". As a result of the review, several issues were identified and discussed with the applicant. In all instances, a path forward for the resolution of these issues was established. The issues discussed with the applicant during the audit were captured for future actions, if necessary (see attached punch list). Details of the audit performed are given in the following section.

UHS Basin and Reactor Service Water Pump-house - Structural Evaluation

The staff reviewed the following documents:

1. U7-UHS-S-CALC-DESN-6002, Revision C, dated March 11, 2011, "Structural Evaluation of the Ultimate Heat Sink and RSW Pump House"

2 U7-UHS-C-CALC-DESN-6003, Revision B, dated January 6, 2011, "Stability Check of Ultimate Heat Sink and RSW Pump House"

Calculation U7-UHS-S-CALC-DESN-6002, Revision C documents the analysis and design of the UHS/PH and consists of a main report and several attachments. The main report describes the design inputs, assumptions, the structural analysis models, the applied loads, load combinations, the concrete design, and composite roof design. The attachments contain the SAP2000 input files, weight computations, comparison of SSI with SAP2000 forces, joint loads from accidental torsion, calculation spreadsheets, various concrete design modules (postprocessors of SAP results), evaluation of soil springs, combination of SAP results with static and dynamic soil springs, etc. The structure consists of the main basin building with the cooling towers located at roof level and the attached pump house enclosing the RSW pump systems. The building is founded on a mat foundation and framed with shear walls. The roof is designed as a composite concrete slab.

The staff reviewed in detail these documents, which includes the analysis and design assumptions, applied loadings and load combinations, thermal analysis, hydrodynamic analysis, the FE models and analyses results, the beam, column, slab and wall design, the lateral earth pressure parameters and loads, the design methodologies, the equivalent static seismic procedures. The following are observations noted:

The analysis is performed with the help of SAP2000 finite element models, using spring, frame, and thick shell finite elements. Eight SAP models are used: four with uniform soil springs, and four for non-uniform soil springs. The four models account for: static loads, dynamic loads, static without vertical loads, and seismic loads without vertical loads. Different soil springs are used under the UHS and the PH. All analysis results are enveloped and seismic forces are typically combined with the square root of the sum of the squares (SRSS) rule for member design.

The software program TEMCO (see description in V&V documents) is used to design sections subjected to thermal gradients and non-thermal axial forces and moments. Thermal gradients are applied to all concrete components like walls, slabs, roof, foundation mat, beams and columns. A total of eight different thermal conditions are applied leading to i.e.: max expansion, max contraction, etc., axial thermal loads are thereby considered in special load combinations. The program assumes a cracked concrete section subject to mechanical and thermal forces and verifies if the resulting strains in the given reinforcement bars and in the given concrete section are within allowable limits.

Inertial seismic forces are applied to the SAP model as equivalent static forces.
The maximum accelerations from the SSI analyses are grouped into 9 vertical groups and 208 panels representing different locations in the structure. Accelerations are averaged within each panel section. Full and empty basin cases are considered, and the dynamic soil spring values are used in the SAP models. Applied inertial forces are computed as the product of the acceleration times the corresponding mass times an adjustment factor (used to compensate for different mesh sizes in SAP and SSI models). X-, Y-, and Z-loads are subsequently combined by the SRSS superposition rule. The X-, Y-and Z- accelerations vary between 0.12g (slab) and 0.60g (tower). It was noted that the maximum element accelerations could be as high as three times the calculated average acceleration for a panel. In order to verify if using average panel accelerations instead of maximum element accelerations is still a conservative approach, section forces (element axial & shear forces, bending moments) taken from the SASSI model were compared with the corresponding SAP values along horizontal and vertical cut lines. Cut lines are defined in Appendix OOO of this calculation. As shown in Appendix BBB-1 of this calculation, which contains the force comparisons, the SAP results are consistently higher than the corresponding SASSI values. Specifically, the comparison for one of the interior NS walls of the cooling tower (cut # 55) showed ratios greater than 1.60 for all six internal forces of the shell element. Based on this example, the overall approach followed for the seismic design is considered conservative as compared with the SSI results.

The water mass in the basin is applied vertically as area load to the base mat. Horizontal pressures due to vertical acceleration are computed as the hydrostatic pressure times the max vertical acceleration of 0.475g. The horizontal pressures due to horizontal acceleration are computed following the methodology in ASCE4-98 and TID-7024 for determining impulsive (rigid) and convective (sloshing) components and accelerations. Horizontal convective loads are applied to walls, buttresses and columns between elevations 41.75’ and 71.00’. The hydrodynamic forces on columns are based on reference 7.62

(Fritz, 1972, “The effects of liquids on the dynamic motions of immersed solids”) of the calculation referenced in item 1 above. Column design for the submerged columns in the basin was originally based on the maximum acceleration along the column axis obtained from the SSI analysis. In the SSI analysis of the UHS, hydrodynamic mass of the submerged portions of the columns were not included. During the audit NINA presented the analysis methodology and results for the UHS Basin columns to account for the missing hydrodynamic mass. In this method new scale factors were used to amplify the horizontal accelerations of the columns. The scale factors were developed from eight decoupled SAP column models (four with additional masses and four without additional water masses) covering all possible boundary conditions and subjected to the envelope response spectra at top and bottom (response spectrum analysis). The scale factors were obtained as the ratio of the resulting nodal acceleration along the column axis with and without additional water masses. The column design was proposed to be revised applying these scale factors to modify the currently used acceleration. Staff found this procedure to be acceptable. The applicant subsequently submitted response 03.08.04-30, Supplement 4, and addressed the issue.

Seismic lateral pressures are computed with both: ASCE4-98 and SSI analyses. All loading diagrams are enveloped and considered. At rest (Ko=0.5) and passive (Kp=1.2) lateral pressures are used, these values are also compatible with the stability analysis.

The design of the concrete elements are performed for the loading combinations from Standard Review Plan (SRP) Section 3.8.4, RG 1.142, ACI 349-97 (which are reduced by eliminating OBE (Eo), wind permutations, pipe breaks and using Ta instead of To), and from ACI350-01. Torsional effects from wind on interior columns, buttresses and walls are considered. Hurricane winds (184 mph) are used for all load combinations including wind loads. No uplift was determined for any load combination.

The calculation references N690-84 for steel design instead of N690-94 which is adopted per section 1.8 for site-specific structures. The applicant agreed to revise the calculations for UHS/PH using N690-1994 (see Punch List item 96 and 235)

Conclusion:

Staff found the assumptions, procedures and methods used to analyze and design the UHS/PH structures to be in accordance with the NRC guidelines and stated descriptions in the FSAR.

UHS Basin and Reactor Service Water Pump-house - Stability Analysis

The staff reviewed U7-UHS-C-CALC-DESN-6003, Revision B, dated January 6, 2011, “Stability Check of Ultimate Heat Sink and RSW Pump House”.

Stability checks are made for floatation, sliding and overturning with the assumption, where conservative, of the empty UHS basin. The loads included the seismic inertia force due to site-specific SSE, design wind speed (134 mph), design basis tornado for Region II (200 mph), and design flood level of 40 ft from MCR dike break.

The floatation factor of safety considering the empty UHS basin with 90 percent dead load and density of water for maximum sediment concentration is calculated as 1.77 which exceeds the minimum specified in the SRP.

The factor of safety for sliding against SSE is calculated assuming the minimum operable water level inside the UHS and the maximum ground water level of 28 ft. The total sliding

force in the N-S direction is the sum of SSE force, soil active force, soil horizontal dynamic force, soil vertical dynamic force, soil surcharge force, soil dynamic surcharge force and hydrodynamic force. The total resisting force is the sum of total friction at the interface of the mat and soil and the total soil resisting force on the passive side of the exterior walls. The factors of safety for sliding in the N-S and E-W directions are calculated as 1.13 and 1.47, respectively. The factors of safety for sliding against design wind and design basis tornado are calculated as 11.5 and 7.2 respectively. These exceed the minimum value of 1.1 specified in the SRP.

The factors of safety against overturning from design wind, design basis tornado, and SSE are calculated as 2.15, 2.11 and 1.47 respectively. These exceed the minimum values specified in the SRP.

Conclusion:

The staff has found that the stability checks for flotation, sliding and overturning have been made properly for the loads and load combinations specified in FSAR, including Table 3H.9-1. It is shown that the factors of safety for flotation, sliding and overturning exceed the acceptable values given in SRP. The maximum Kp value required to yield acceptable sliding factor of safety is 1.2 which is less than the limit of 3.0 given Reference 12 (page 56 of 100) of the calculation.

UHS Cooling Tower Tornado Missile Design (Ref. 10, Section 5.9, Page 494 of 618)

The tornado missile spectrum used is from RG 1.76 for Region II. The cooling tower fan cell is protected from tornado missiles by four panels of steel beams and two layers of grating. Since the missile protection system is located at El. 153’ 0’’, automobile missile need not be considered; only steel sphere and schedule 40 pipe missiles are to be included in the design. The grating is such that the sphere missile will not pass through to impact the fans. The grating and the beams are designed to withstand the impact of tornado missiles (local damage and overall response). For local missile damage evaluation, Ballistic Research Lab. (BRL) formula for penetration into steel is used. The concept of plastic impact is utilized wherein the impact energy is compared with the plastic capacity for flexure or shear.

The supply/return missile protection hoods are designed as a two-way slab spanning in both directions. The concrete is designed per ACI 349-97. The slab is 2 ft in thickness.

SRP Section 3.5.3 specifies the minimum thickness of wall (14.3 in) and roof (10.4 in) for concrete design strength of 4000 psi. The UHS and RSW pump house walls and roofs have thicknesses exceeding these values. For example, pump house wall is 72 inches. The pump house roof is 24 inches thick.

The impact load from automobile missile including dynamic load factor is 900 Kips. The panel capacity considering shear and flexure is calculated as 1772 Kips which exceeds the impact load.

For roof slab, both the missile scabbing and punching shear checks are made.

The overall damage to concrete barriers in empty UHS basin is also evaluated for both local response and overall response.

Conclusion:

The tornado missile design of UHS cooling tower has considered the tornado missile spectrum per RG 1.76, Region II; the cooling tower cell fans are protected from missiles by steel grating. The pump house wall and roof have concrete thicknesses that exceed the minimum thickness specified in SRP Section 3.5.3. The pump house is also designed to withstand the impact from automobile missile for both local response and overall response. The staff confirmed that the tornado missile design of UHS cooling tower conforms to the FSAR criteria and meets the SRP requirements.

Diesel Generator Fuel Oil Tunnels (DGFOT) - Analysis and Design

This review involves the confirmation of seismic input from SSI, seismic and wind and tornado loadings, flood protection, seismic wave propagation effects, seismic gap, and stability evaluation). The following calculations and reports are reviewed:

1. U7-YARD-S-CALC-DESN-6006, Revision B, “Basic Structural Design of Diesel Generator Fuel Oil Tunnels (DGFOT)”
2. U7-YARD-C-CALC-DESN-6002, Revision B, “DGFOSVT SSI Analysis”
3. CALC-EORA-002, Revision 4, (MACTEC report), “Ultimate Static and Dynamic Coefficients of Sliding Friction”
4. CALC-EORA-001, Revision 2, (MACTEC report), “Foundation Springs (Seismic) for Category I and other Structures”

The staff reviewed calculation U7-YARD-S-CALC-DESN-6006, Revision B, “Basic Structural Design of Diesel Generator Fuel Oil Tunnels (DGFOT)”. The calculation describes the structure, the input parameters and the analysis and design of the DGFOT. The concrete structure consists of the main buried tunnel and the end access regions connecting the tunnel to the DGFOSV and RB. Walls and slabs are at least 2 ft thick. Typical tunnel cross section is 7.5 ftx11 ft, with the access shafts emerging 15ft above ground level. Only the shortest of three tunnels (L=49 ft) was analyzed.

During the audit review the staff noted that:

A restraint is required around the access regions to prevent movement from a horizontal missile hit. The support will be designed as to not restrain seismic or settlement induced displacements. The support will be attached to the RB and to the DGFOSV and will be designed during the detailed design phase.

The structure is simulated with 12 SAP models (10 static and 2 dynamic models), using shell elements and soil springs (uniform and non-uniform or pseudo-coupled, and soft / stiff soils). Concrete design is performed per ACI 349-97 load combinations.

Design accelerations are taken from calculation number **U7-YARD-C-CALC-DESN-6002, Revision B,** “DGFOSVT SSI Analysis”. The inertial loads are applied as uniform area loads or point loads in X-, Y-, and Z- directions, thereby elements are grouped according to location and type. A comparison is made between the global resulting base shear and total vertical force applied to the SAP model, with the same quantities obtained from the SASSI model. If the SAP resultant values are less than the corresponding SASSI values, adjustment factors are defined to amplify the equivalent accelerations used in the SAP model. In this case and in order to match SSI and SAP results, SSI accelerations were amplified by a factor of 1.15. The final accelerations are as follows:

	Horizontal	Vertical
Below ground	0.45g	0.37g
Above ground	0.85g	0.40g

The resulting maximum penetration of tornado missiles into the wall was calculated to be 15", thus less than the 24'' nominal wall thickness. The global impact effects resulted in added torsional reinforcement in the tunnel section, and in an added support frame at the access shafts to maintain stability.

Seismic wave propagation effects are based on the ASCE4-98, Section 3.5, procedures. The friction coefficients are taken from MACTEC report "Ultimate Static and Dynamic Coefficients of Sliding Friction", CALC-EORA-002, Revision 4. The resultant total strain of 1.83×10^{-4} leads to allowable (less than 0.9) concrete and reinforcing steel interaction values. Forces (axial, moment and shear) in bends were computed following Reference 7.31 of calculation referenced in item 1 above (Goodling, 1983, "Buried Piping – An Analysis procedure Update") for three situations representing different tunnel alignments. Horizontal spring values are taken from Reference 7.12 of calculation referenced in item 1 above (MACTEC report CALC-EORA-001, Revision 2, "Foundation Springs (Seismic) for Cat I and other Structures"). On bend #3 the computed horizontal shear force acting on the bend leg (L=4 ft) exceeded the resisting soil capacity, therefore reduced bend forces were calculated using the maximum (Kp=3) passive pressure present at the tunnel basemat as the maximum soil resistance. The final enveloping design forces were applied to the tunnel section nodes of the SAP model as follows: bending moment applied as tension/compression to the walls; horizontal shear applied to the top/bottom slab and axial force as tension/compression on the whole perimeter. The applicant will revise this calcs based on a propagation velocity of 3000 fps as agreed upon during the audit.

Conclusion:

As a conclusion, staff confirmed assumptions, procedures and methods used to analyze and design the DGFOT structure to be adequate. Revisions of the report are still pending to incorporate the effects of the wave propagation velocity. (Punch List Item 82)

Diesel Generator Fuel Oil Tunnels (DGFOT) - Stability Evaluation

The staff reviewed Calculation U7-YARD-S-CALC-DESN-6003, Revision A, April 10, 2010, "Diesel Generator Fuel Oil Storage Vault Tunnels Stability Evaluation".

The tunnel is 7.5ft wide and 11.0 ft deep; the top of the slab is at grade. Ground water table is correctly taken as 28'-0" MSL.

Wind and tornado will not affect this tunnel. Only seismic induced instability and flotation under DBF are analyzed. The DBF is the breach of Main Cooling Reservoir leading to 40 ft MSL flood level.

Kinetic friction coefficient is 2/3 of the static coefficient (0.58); internal angle of soil friction is 30 deg.

Amplified response spectra due to site-specific SSE (0.13g) is used.

The Elastic Solution Method is used for dynamic lateral soil pressure with at-rest conditions. This is described in ASCE 4-98. Equations for sliding force and overturning moment are given in this document.

The active pressure condition is considered for the stability check of the DGFOT if necessary. When considering sliding and overturning due to SSE, the Mononobe-Okabe solution is used for active pressure conditions. Equations for lateral soil load due to horizontal earthquake excitation above ground water table, lateral soil load due to horizontal earthquake excitation below ground water table, lateral soil load due to vertical earthquake excitation above ground water table, soil load due to vertical earthquake excitation below ground water table, and hydrodynamic load are given in ASCE 4-98. Foundation stability is evaluated in two steps. Step A assumes no displacement at the base. Therefore, the load is calculated considering at-rest conditions. The resistance against sliding is based on the available friction at the base of the structure. The restoring moment against the overturning is based on the self-weight of the structure. If the calculated factor of safety is below the minimum acceptable value, the structure begins to displace and engage the active and passive pressures. Step B calculates the factor of safety considering displacement at the base. Note that the more the structure displaces, the more passive pressure is engaged until the maximum passive pressure is reached. Therefore, the structure is checked to ensure that Kp less than or equal to the maximum value of 3.

The factors of safety are obtained as below:

Floatation = 1.6
SSE Sliding = 1.11 (Kp =0.53)
SSE Overturning = 1.10 (Kp =0.79)

Conclusion:

The stability checks for flotation, sliding and overturning have been made properly for the applicable loads (amplified response from site-specific SSE and Design Basis Flood from MCR break) and load combinations specified in FSAR and FSAR Table 3H.9-1. It is shown that the factors of safety for flotation, sliding and overturning exceed the acceptable values given in SRP. The maximum Kp value required to yield acceptable overturning factor of safety is 0.79 which is less than the limit of 3.0.

Outstanding DGFOSV issues (Acceleration, amplification of vertical acceleration, evaluation)

The staff reviewed calculation U7-YARD-C-CALC-DESN-6001, Revision B, “Diesel Generator Fuel Oil Storage Vaults, Soil-Structure-Interaction Analysis”, for the acceleration values imported into the SAP model for the vault. Maximum absolute accelerations for about 1500 nodes in walls and roof slab range from:

X-direction=0.26g to 0.33g
Y-direction=0.26g to 0.34g
Z-direction=0.30g to 0.54g (node #7753 – roof slab at 30’)

In calculation U7-YARD-S-CALC-DESN-6001, Revision B, 0.86g is used as vertical acceleration for the design of the buried roof slab (1.9*0.30*1.5=0.86g), considering a slab frequency of 12 Hz, and the 0.30 RG1.60 vertical ground spectrum:

This value is higher than the Z-direction SSI range described above, therefore the design acceleration for the roof slab is bounding. The remaining static equivalent accelerations used for design of the walls and slab were obtained and transferred from the approved SSI analysis, and are therefore adequate. In order to match the total horizontal seismic base shear and vertical load from the equivalent static analysis (SAP) with the corresponding values from the SSI (SASSI) analysis, a scaling factor was used to adjust the resulting total vertical inertial load. This procedure is deemed adequate.

Radwaste Building - Structural Evaluation

The staff reviewed the following documents:

1. U7-RWB-S-CALC-DESN-6005, Revision C dated November 10, 2010, "Radwaste Building Structural Evaluation"
2. U7-RWB-S-GDD-6001, "Structural Design Criteria for Radwaste Building"
3. U7-RWB-S-CALC-DESN-6003, Revision A, "Radwaste Building Stability Evaluations (Licensing)" April 26, 2010.

At the time of the audit when Reference 2 was reviewed, RWB was categorized as RW-IIb (hazardous) in accordance with RG 1.143, Revision 2. However, the RWB is conservatively designed for earthquake, tornado and wind loadings consistent with guidance in classification RW-IIa described in RG 1.143, Revision 2. The RWB is located approximately 20 feet west of the RB. Therefore, it is also designed to meet Seismic II/I criteria; in fact, it is designed for the same ABWR DCD SSE and tornado loadings applicable to seismic Category I structures.

Design Basis Wind Load: for Seismic II/I criteria, use the DCD requirements per ASCE 7-88 (50 yr wind speed and I = 1.11); per RG 1.143 RW-IIa criteria, use the 50 year wind speed of 126 mph and I= 1.15). The RG 1.143 requirement governs.

Design Basis Earthquake: per RG 1.143 RW-IIa criteria, use RG 1.60 response spectra anchored to 0.15g.

The staff requested the applicant to clarify in FSAR Table 3H.9-1 that the seismic input spectra used for design of RWB should be corresponding to 4 percent damping since the design is being done for ½ SSE per RG 1.143. The applicant acknowledged and will revise this table in next FSAR revision.

Extreme Environmental Loads:

Seismic II/I criteria: envelope of amplified site-specific SSE and 0.3g RG 1.60 spectra. RSA is performed using the SAP2000 model.

External Flood: DBF for MCR breach of 40 ft MSL for Seismic II/I design

Tornado Loads: Seismic II/I criteria: use DCD tornado parameters.

For design: use RG 1.143 RW – IIa criteria; 3/5 of Region I tornado parameters in RG 1.76 or ANS 2.3 tornado at a probability of 10^{-5}/yr.

For tornado design, the II/I criteria governs.

Tornado Missile Loads: Seismic II/I criteria require that the RWB not collapse on the RB under tornado loads. Therefore, the RWB exterior walls must be designed to withstand the overall effects of tornado winds W_w in combination with tornado missile loads W_m. Although

structural panels need not be designed for local missile impact, the panel supports and the overall structure are required to support these missile strikes elastically.

Design flood level:

The staff asked the applicant to clarify why the design flood level used for design of RWB was taken to be 33 ft MSL while stability evaluation and design for II/I evaluation considered flood elevation of 40 ft MSL. The applicant explained that based on RG 1.143, Revision 2, classification of the RWB as RW-IIb, the building need not be designed for protection against design basis flood of 40 ft MSL, which was described in FSAR Section 2.4S.13.2. However, to ensure that the RWB does not have any interaction with adjacent category I buildings, the stability and II/I design of the building assumed 40 ft MSL design basis flood for the site.

Conclusion:

The focus of this audit was to verify whether the applicant's design calculations have properly used the FSAR criteria for loads and load combinations. Some of the design parameters of RWB meets and exceeds the requirements of RW-IIb in RG 1.143, Revision 2, which is the licensing basis of STP Units 3 and 4 at the time of the audit.

Radwaste Building - Design and Stability Evaluation

In Reference 3 **(**U7-RWB-S-CALC-DESN-6003, Revision A), the RWB is described as a reinforced concrete structure consisting of walls and slabs supported by a mat foundation. Liquid radwaste storage tanks are housed inside concrete cubicles located at the basemat elevation, which is approximately 57 ft below grade. These cubicles are lined with steel liner plates to eliminate migration of any liquid outside of the concrete cubicles. The RWB is approximately 291'-5" by 127'-7", extending approximately 61 ft above grade and 57 ft below grade.

Stability Evaluation

Roof decking is at El. 95'0" MSL The top of the base mat is at El (-) 11'0'' MSL. The lower bound weight of the RWB is 174160 kips is used in the stability calculations for wind and tornado loads.

The soil pressure coefficients used are:

Active soil pressure coefficient = 0.333
At-rest soil pressure coefficient = 0.50
Passive pressure coefficient = 3.0
Static coefficient of friction = 0.58

Loads

At the time of the audit, the RWB was designed to RG 1.143, (Revision 2), classification RW-IIb. However, some of the design parameters of the RWB were designed to RW-IIa conditions, which are beyond the design basis conditions of RW-IIb. For example, the RWB is designed for tornado loads in accordance with RW-IIa requirements of RG 1.143, Revision 2. The structure is also designed for tornado loads according to ANS 2.3 with a probability of $1x10^{-5}$ /yr or three-fifths of the criteria in RG 1.76, Revision 1. (i.e., 3/5 of 230 mph for Region I). The RWB is also required to meet the Seismic II/I criteria of the DCD due to its proximity to the seismic Category I structures. Therefore, the maximum tornado wind load resulting from these three sources is used in the design of RWB. DCD tornado

governs for tornado loads on the RWB. The governing tornado differential pressure is from DCD tornado i.e., 2 psi.

The RWB design criteria requires wind loads be calculated based on the wind velocity pressure from ASCE 7-88 or ASCE 7-95, whichever is larger. ASCE 7-95 governs and conforms to RG 1.143 Revision 2.

Stability Checks

In Reference 3, stability checks for floatation during design basis flood, for sliding against design basis wind, design basis tornado and seismic loads and for overturning due to wind, tornado and seismic loads are made.

Floatation

The dead weight used contains the self-weight of concrete walls and slabs, steel trusses, and bullet resistant enclosures. Conservatively, the tank, crane, and equipment weights are ignored. Only 90 percent of the dead weight of the structure is considered to resist the buoyancy force. The calculated factor of safety against flotation from Design Basis Flood is 1.5; this exceeds the minimum value of 1.1 specified in the SRP.

Sliding

The sliding resistance is calculated as the coefficient of friction times the lower bound weight of the structure reduced by the buoyancy force from the water table. The safety factor against sliding due to design basis wind is calculated as the ratio of the sliding resistance to the wind force acting on the structure is obtained as 52. Using the same approach, the factor of safety for sliding due to design basis tornado is calculated as 36.

For the calculation of factor of safety against sliding due to seismic loads, the following approach is used. The driving force is calculated by summing the horizontal seismic forces and dynamic soil forces in the applicable direction. The friction force is calculated by first determining the upward buoyant force due to ground water. Two-thirds of the friction coefficient is used in calculating the friction force. The safety factor is obtained as 1.92 which is the minimum for different permutations of 100-40-40 combinations. The minimum K_p to ensure a safety factor of 1.1 is calculated as 0.542.

Overturning

The factor of safety against overturning is calculated as the ratio of the resisting moment to the overturning moment due to the imposed load in that direction from design wind, design basis tornado, and SSE. The factors of safety against overturning are 78 (for design wind), 53 (for design basis tornado) and 4.3 (SSE).

During the audit the staff had expressed that the amplified ground motion for RWB computed at the foundation level may not be conservative, since the motion at ground surface may be higher. The applicant will address this in a supplemental response to RAI 03.07.02-13, and will revise the stability calculations using amplified motion at the ground surface.

Conclusion:

The staff has found that the stability checks for flotation, sliding and overturning have been made properly for the loads and load combinations specified in FSAR and are consistent with FSAR Table 3H.9-1. It is shown that the factors of safety for flotation, sliding and overturning exceed the acceptable values given in SRP. The maximum Kp value required

to yield acceptable sliding factor of safety is 1.2 which is less than the limit of 3.0 given Reference 2 (page 56 of 100) of the calculation identified in item 3 above.

Stability calculations for other II/I structures (SB, CBA, and TB)

Stability Evaluation of Service Building (SB)

The staff reviewed Calculation U3-SB-S-CALC-DESN-2100, Revision C dated October 14, 2010, "Calculation for Service Building Stability Factors of Safety". Particular attention is on the safety factor reported in FSAR Table 3H.6-14

SB is a non-seismic Category I structure with a potential for II/I interaction with other seismic Category I buildings. It is located east of the control building and the RB. Factors of safety against flotation, sliding and overturning are calculated using static methods of analysis 4. The staff asked the applicant if the input ground motion for II/I design of the SB should consider enveloping the amplified site-specific spectra for SB with the RG 1.60 spectra anchored at 0.3g used by the applicant to take into account effect of adjacent heavy structures. The applicant decided to take this approach and will revise FSAR Table 3H.9-1 accordingly.

The safety factor against floatation due to the design basis flood from MCR dike breach is calculated as 1.4. The minimum factors of safety against overturning and sliding are 2.65 and 1.81. These exceed the SRP minimum values. The results of stability checks are included in FSAR Table 3H.6-14, Revision 6

Conclusion:

The staff has confirmed that the stability checks for flotation, sliding and overturning have been made properly for the loads and load combinations specified in FSAR and FSAR Table 3H.9-1. It is shown that the factors of safety for flotation, sliding and overturning exceed the minimum values given in SRP.

Stability Evaluation of Control Building Annex (CBA)

The staff reviewed calculation U7-CBA-C-CALC-DESN-6001, Revision B, "Control Building Annex Stability Evaluation." During the audit held in March, 2011 (ML111320094), the staff had identified that the calculation had erroneously included a statement that the stability evaluation was applicable to DCD standard plant, and the applicant had agreed to revise the calculation to remove the statement. Staff verified that the calculation has been revised to remove the statement. The stability evaluation of the CBA was performed using a preliminary design of the structure with dimensions 82.09 ft (L) x 77.22 ft (W) x 24.8 ft (H). The structure is surface mounted with a 4 ft thick basemat with the top of basemat at elevation 35 ft MSL. Stability evaluation was performed for wind, tornado, seismic and flood loading. The factors of safety against floatation, sliding and overturning were 1.19, 1.16, and 2.03 respectively. These values are all within the allowable limits of SRP Section 3.8.5, and considered acceptable. Seismic loads were evaluated considering the structure as a single degree of freedom system. Flood loads were based on PMF level of 40 ft MSL. It was noted that the wind loads were based on the provisions of ASCE 7-88 instead of the newer ASCE 7-05 referenced in SRP Section 3.3.1. Also, it was noted that live loads were included in calculating the stabilizing forces for stability evaluation. This was brought to the attention of the applicant. The applicant subsequently submitted response 03.07.02-13 S3, and stated that the calculation was revised to address the issues and that the calculated

factors of safety for sliding and overturning were not adversely impacted by the change. (Punch List Item 101)

Stability Evaluation of Turbine Building (TB)

The staff reviewed Calculation U3-TB-S-CALC-DESN-2100, Revision B, “Calculation for Turbine Building Stability Factors of Safety.”

TB is a non-seismic Category I structure with potential for interaction with the Control Building, which is a seismic Category I structure. It is located immediately north of the control building. Stability evaluation was performed for seismic, wind, tornado, and floatation for flood level of 40 ft MSL. Seismic loading was based on response spectrum analysis of the stick model of the TB using site specific SSE as the input ground motion. Wind loading was considered for site-specific design wind speed of 134 mph using ASCE 7-05 guidance. Tornado wind was considered using site-specific tornado wind speed of 200 mph. Reduction of tornado wind pressure was considered using the guidance in Bechtel topical report BC-TOP-3-A, “Tornado and extreme Wind Criteria for Nuclear Power Plants,” which was accepted for use by the NRC. A sliding coefficient of friction of 0.3 was used per FSAR Table 2.5S.4-16 for stratum D soil. Several supporting calculations were referred for development of mass and stiffness properties of the stick model of the TB used for determination of seismic demand.

The staff asked the applicant to explain the basis for using the site-specific SSE for stability evaluation of the TB and the SB, and not considering potential amplification of input motion due to the presence of adjacent structures. The applicant explained that since the TB was relatively heavy compared to the adjacent structures, any significant amplification of input motion due to presence of adjacent structures was not expected. However, the applicant decided to use amplified site-specific SSE for stability evaluation of the SB since the SB is a relatively light structure. FSAR Table 3H.9-1 will be revised accordingly.

The staff confirmed the methodology used for stability determination including computation of dead load, earth pressures, wind load, tornado loads, seismic loads and floatation. The minimum factors of safety for sliding, overturning and floatation were within the limits of SRP Section 3.7.5. However, it was not clear how the mass and stiffness properties of the stick model used for seismic analysis were derived from the various calculations referred in the calculation. Also, it was not clear how the seismic demand was calculated considering three directional earthquakes. The applicant was asked to clarify these. Also an error was detected in one of the referenced calculation number. The applicant will address these issues in a supplemental response to RAI 03.07.02-13. (Punch List Item 90)

RSWT wave propagation

During the audit held in March, 2011 (ML111320094), it was noted that the apparent wave velocity used for calculating seismic wave propagation effects on RSW piping tunnels was 6000 ft/sec, whereas the FSAR stated that a value of 3000 ft/sec is used. This was discussed further with the applicant during this audit and it was also pointed out that the design did not account for the additional lateral pressure on tunnel walls at bends. The applicant committed to determine seismic wave propagation effects using apparent wave velocity of 3000 ft/sec and maximum ground velocity based on site-specific SSE. The applicant also proposed to use a triangular pressure distribution at tunnel bend limited by the maximum passive pressure and revise COLA accordingly. The applicant will submit supplemental response to 03.08.04-30 to address the issues.

3C. Miscellaneous Issues

Friction coefficients (3.8-8)

The issue of coefficient of friction was discussed with the applicant during the audit:

The minimum coefficient of friction needed to demonstrate stability of the RB and the Control building (CB) was 0.47, which was the value of dynamic coefficient of friction and was taken as 2/3 of 0.7 based on shear capacity of soil. During the audit held in March, 2011 (ML111320094), the staff had brought to the attention of the applicant that the static coefficient of friction assumed at other interfaces of the foundation, e.g., in the water-proof membrane or concrete to concrete interface was 0.6. Therefore, sliding may occur at these interfaces before exceeding soil shear capacity of 0.7. In its response 03.08.04-19 S1 in April 2011, the applicant addressed the issues by requiring a static coefficient of friction of 0.75 minimum for both the water-proof membrane and the concrete interface. The inspections, tests, analyses, and acceptance criteria for the water-proof membrane was revised accordingly in the FSAR mark-up submitted with the response. However, the applicant did not include in the FSAR mark-up the requirement for intentional roughening of concrete needed to achieve the minimum 0.75 coefficient of friction. The applicant agreed to include the requirement in the FSAR and will provide FSAR mark-up in its supplemental response to 03.08.04-19.

Evaluation of soil bearing pressures (3.8-9)

The subject of computing bearing pressures under foundation basemat by considering an equivalent uniformly loaded area and its relevance to design of foundation was discussed with the applicant. This was the subject of RAI 03.08.04-35. The applicant explained that the above method of computation was done according to the recommendations in industry recognized text books and publications, such those Joseph E. Bowles, J. B. Hansen, and G. G. Meyerhof, and provides a consistent method of comparing the actual bearing pressure and the calculated ultimate bearing capacity under a foundation subjected to vertical load and moment. The design of the basemat is done using finite element method using foundation soil springs that takes into account the effect of variation of pressure distribution under the foundation under vertical load and moment. The staff agreed with the explanation and the applicant will revise response to RAI 03.08.04-35 to document this.

Wind/hurricane/tornado loadings (3.8-11)

The subject of using Importance Factor (IF) of 1.0 with 100-year return period wind was discussed with the applicant. The applicant explained that though a literal interpretation of SRP Section 2.3.1 and SRP Section 3.3.1 may imply that an IF of 1.15 should be used with 100-year return period wind speed, use of the value 1.0 is consistent with ASCE 7 standard that is referenced in both SRP. The staff also noted that the same interpretation as presented by the applicant was used for economic simplified boiling-water reactor design certification. The applicant agreed to provide justification for using IF of 1.0 with 100-year wind speed in its response to RAI 03.08.04-30, and will revise FSAR Section 3H.6.4.3.2 accordingly.

Beam shear (3.8-21)

The subject of averaging of out-of-plane shear force for all elements across the entire width of a slab was first identified during the audit held in October, 2010 (ML110110104), and the staff subsequently issued RAI 03.08.04-34. The subject was further discussed with the applicant during the audit held in March, 2011, when the applicant agreed to not using the averaging of shear forces obtained from finite element analysis across the entire width of slab, and provided response to RAI 03.08.04-34 with a plan for revising the calculations and designs. The response was discussed with the applicant during the audit, and it was pointed out that FSAR text and design results reported in the FSAR will need to be updated. The applicant will address this in the supplemental response to RAI 03.08.04-34.

Computer Code V&V SAFE and PCACOLUMN

In the March audit regarding the SAFE program, the "Release Memo" warns users about the 30 percent deviation from "exact" values observed in one of the validation examples. The staff verified the pop-up memo that comes up when SAFE is started, which includes an awareness call and explanation of the observed difference and a statement about the acceptability of certain deviations.

During the review of the in house V&V documentation performed during the previous audit, it was noted that PCACOLUMN, Version 4.10, does not cover the ACI 349-97 specification. S&L explained that every user of the program is alerted about this limitation and that it is the operator's responsibility to evaluate in the design calculation the impact of the special requirements resulting from ACI 349-97.

4. Audit Findings

There is no audit finding or observations. NINA/S&L committed to the following actions, which include additional analysis, submitting revised or supplemental responses to existing RAIs, and updating the FSAR:

1. In addressing the DNFSB discovery of the problems with SASSI, NINA needs to address the V&V) of modified MSM if MSM is used to validate SM.

2. For the UHS/RSW Pump House analysis, need clarification on the pressure distribution for the calculated seismic soil pressures on the RSW pump house wall.

3. For the DGFOSV, clarification is needed why two different input motions are developed and used in separate DGFOSV SSI analyses.

4. The design basis of ground water elevation needs to be clearly established and applied clearly and consistently without confusion.

5. The issue of wave propagation for buried structure remains unclear and need clarification.

6. For the TB stability analysis, it was not clear how the seismic demand was calculated. Clarification is also needed how the mass and stiffness of the stick model used for the seismic analysis was derived.

7. The RWB stability calculation should consider amplified motion at ground surface.

8. Address the use of Important Factor (IF) to clarify departure from SRP Section 3.3 guidance.

9. The subject of averaging of out-of-plane shear force for all elements across the entire width of a slab was identified in previous audits and need to be resolved. The applicant has committed not using the averaging of shear forces obtained from finite element analysis across the entire width of slab.

5. Conclusion

The tentative schedule to provide draft responses and feedback to staff is mid-August and to finalize all input by the end of August 2012. There is no technical issue identified in this audit. The actions summarized above were agreed to by NINA and S&L management during the exit. The staff agreed to resume the weekly open item call for schedule and status update.