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July 27, 2011
U7-C-NINA-NRC-110103

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
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South Texas Project
Units 3 and 4
Docket Nos. 52-012 and 52-013
Supplemental Response to Request for Additional Information

During an audit on May 23-27, 2011, the NRC Staff requested that Nuclear Innovation North America LLC (NINA) provide additional information to support the review of the Combined License Application (COLA). Attached are supplemental responses to NRC staff questions included in Request for Additional Information (RAI) related to COLA Part 2, Tier 2, Sections 3.7 and 3.8. The attachments provide supplemental responses to the RAI questions listed below:

03.07.02-13
03.08.04-18
03.08.04-33

Where there are COLA markups, they will be made at the first routine COLA update following NRC acceptance of the RAI response.

There are no commitments in this letter.

If you have any questions regarding these responses, please contact me at (361) 972-7136 or Bill Mookhoek at (361) 972-7274.

D091
NRO

I declare under penalty of perjury that the foregoing is true and correct.

Executed on 7/27/11

Scott Head
Manager, Regulatory Affairs
South Texas Project Units 3 & 4

jep

Attachments:

RAI 03.07.02-13, Supplement 3
RAI 03.08.04-18, Supplement 3
RAI 03.08.04-33, Supplement 1

cc: w/o attachment except*
(paper copy)

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RAI 03.07.02-13, Supplement 3

QUESTION:

(Follow-up Question to RAI 03.07.02-1)

With regard to Item c of the response to RAI 03.07.01-13, the applicant is requested to address the following:

1. The FSAR mark-up in the response to item (b) of RAI 03.07.02-1, did not include the list of non-Category I structures requiring the enhanced seismic design and analysis. The applicant is requested to include in FSAR 3.7.2.8 the five identified non-Category I structures that could interact with the Category I structures.

2. The response to item (c) of RAI 03.07.02-1 indicated that non-Category I structures with the potential to interact with Category I structures have not yet progressed to a point where sliding and overturning potential as a result of the SSE can be evaluated. However, as identified in SRP guidance 3.7.2I.8., the staff must review the applicant's seismic design of these non- Category I structures. As such, the applicant is requested to provide in the FSAR factors of safety against sliding and overturning including the basis of coefficient of friction used in the analysis during an SSE for Turbine Building, Radwaste Building, Service Building, Control Building Annex, and Plant Stack.

SUPPLEMENTAL RESPONSE:

The Supplement 2 response to this RAI was submitted with Nuclear Innovation North America (NINA) letter U7-C-NINA-NRC-110066, dated April 25, 2011. This supplement provides the response to the following action items discussed in the NRC audit performed during the week of May 23, 2011.

a. *Clearly describe in the FSAR how seismic demand for non-seismic II/I structures for stability evaluation is determined (Clarification Issue 3, Punch List Item 14)*

   See revised COLA Sections 3.7.2.8 and 3.7.3.16 in Enclosure 1.

b. *Revise RWB stability calculation considering amplified motion at ground surface and revise COLA as necessary (Audit Action Item 3.7-39, Punch List Item 73)*

   The Radwaste Building (RWB) stability calculation has been revised using the amplified site-specific SSE motions at ground surface. These amplified motions are shown in new COLA Figures 3.7-44 through 3.7-46 in Enclosure 1. The revised sliding and overturning factors of safety are provided in revised Table 3H.6-14 in Enclosure 1.

c. *Clarify title for Figure 3H.6-137 to specify that it is applicable to Category I site-specific structures (Clarification Issue 17.5, Audit Action Item 3.8-33, Punch List Item 81)*

The title of Figure 3H.6-137 has been revised to specify that the figure is applicable to stability evaluation of Category I site-specific structures. See Enclosure 1 for revised Figure 3H.6-137.

d. *Add the factor of safety for flotation in Table 3H.6-14 (Audit Action Item 3.7-43, Punch List Item 88)*

COLA Table 3H.6-14 has been revised to include flotation safety factors. See Enclosure 1 for revised Table 3H.6-14.

e. *Turbine Building Seismic Calculation, Fluor calculation number U3-TB-S-CALC-DESN-2100 Rev B, should be revised for the following (Audit Action Item 3.7-45, Punch List Item 90):*

1) *Assumption 1 on sheet 9 of 288 should be clarified to clearly describe how mass and stiffnesses were derived from different Turbine Building models*

Assumption 1 will be revised to read as follows:

1. The stiffness used for the Turbine Building in this calculation is the stiffness as calculated in Calculation U7-TB-S-CALC-DESN-2001 Rev 0. The methodology used to determine the stiffness is described in Calculation U7-TB-S-CALC-DESN-2001 Rev 0, Section 4.0 with further explanations given under Section 4.0 Items 4 and 5. The masses used for the Turbine Building are described in section 5.15 of this calculation. Separate full scale RISA3D models are used to determine the overall mass. These separate models are contained in Appendices 01, 02, 03 and 05 of this calculation. The consolidation of mass from these models to the 5 primary stick model elevations indicated in Calculation U7-TB-S-CALC-DESN-2001 Rev 0 is contained in Appendix 09 of this calculation.

2) *On sheet 8 of 288, correct Reference 3 document number from U3-TB-S-CALC-DESN-2001 to U7-TB-S-CALC-DESN-2001*

Reference 3 will be revised to read as follows:

3. Fluor Calculations:

a. Calculation for Turbine Building Mass & Stiffness Model, U7-TB-S-CALC-DESN-2001 Rev. 0
b. Calculation for STG Pedestal Mass & Stiffness Model, U7-TB-S-CALC-DESN-2002 Rev. 0
c. Calculation for Turbine Building Vertical Mass & Stiffness Model,

U7-TB-S-CALC-DESN-2005 Rev. A

d. Calculation for STG Pedestal Vertical Mass & Stiffness Model, U7-TB-S-CALC-DESN-2006 Rev. A

Fluor Documents:

a. General Design Document - Structural Engineering Criteria – Turbine Building, U7-PROJ-S-GDD-2911 Rev. E
b. Unit 3 Turbine Building Structural Loading Document, U3-TB-S-CALC-DESN-2001 Rev. A

3) *Revise Section 5.15 to clearly describe how the seismic demand was determined. For example, clarify the statement in the last paragraph of sheet 250 "after running the RSA static analysis is then run with the RSA results to determine the base shear". Also describe how the RISA analysis was done in 3 directions and how the 3 directional responses were combined*

The following will be added to Section 5.15 to provide further clarification on the derivation of the seismic demand:

For each orthogonal direction, N-S, E-W and Vertical, of the Turbine Building, a simplified stick model was developed which represents the mass and stiffness characteristics of the Turbine Building in that particular direction. Utilizing the corresponding directional 5% damped site-specific Safe Shutdown Earthquake (SSE) response spectra, as determined in Calculation U7-SITE-C-CALC-DESN-6007 Rev B, a Response Spectrum Analysis (RSA) was performed. In this analysis, a 5% damping ratio and the Complete Quadratic Combination (CQC) modal combination method was used. This analysis was run on the three individual orthogonal direction stick models to determine the corresponding seismic base shear for that particular orthogonal direction. Since the mass assigned to the base is fixed in all degrees of freedom, its response is not included in the RSA results. A hand calculation multiplying the zero-period-acceleration (ZPA) of the site-specific SSE and the mass at the base is then added to the RSA results to determine the overall Turbine Building seismic demand for that particular orthogonal direction. The same procedures were used for the turbine generator pedestal stick models, as it shares a common basemat with the Turbine Building, and thus is part of the overall seismic demand for the Turbine Building.

To determine the overall seismic demand for the Turbine Building, the three orthogonal seismic demands were spatially combined using the 100-40-40 rule as outlined in Regulatory Guide 1.92 "Combining Modal Responses and Spatial Components in Seismic Response Analysis" Rev 2 . Further details are shown in the Load Combination Permutation Matrix outline in Section 5.17 of this Calculation.

For COLA revisions as a result of this part of the response see mark-up to COLA Sections 3.7.2.8 and 3.7.3.16 in Enclosure 1.

f. For Service Building stability calculation (U3-SB-S-CALC-DESN-2100 Rev. B), expand Section 5.14 to fully describe how the seismic analysis was performed and specifically address the following (Audit Action Item 3.7-47, Punch List Item 97):

1) How the stick model mass and stiffness were calculated or provide a copy of Ref. No. 23, U3-SB-CALC-DESN-2001 Rev. 0, "Calculation of Service Building Mass and Stiffness Model" for review.

In order to first determine the mass and stiffness of the Service Building, RISA-3D was the software chosen to perform this analysis. To create the preliminary analysis model in RISA-3D, all major structural elements of the Frameworks Plus and Bentley Structural models were exported and then imported into RISA-3D. These elements included steel columns, steel beams, concrete shear walls and diaphragms, and concrete basemats.

In addition to the structural geometry, parameters such as member-to-member connectivity and support restraints are assigned within RISA-3D. Member to member connectivity is to account for the type of connection between members and whether shear, axial or moment transfers between members is allowed. Support restraints are used to calculate the reactions to a supporting structure such as a basemat or earth that the structure has. Diaphragms are added to the analysis model to account for floor slabs that may not be modeled in the structure. For example, the weight of the floor slab may be added to the support steel to account for the mass, while to account for the stiffness of the slab itself a diaphragm will be assigned to all members within the slab area. The connectivity and characteristics of the diaphragm are also assigned.

Utilizing the geometry, member connectivity, member attributes, and material properties a 3D stiffness matrix is generated by RISA-3D. This 3D stiffness matrix accounts for torsional bending, axial and shear stiffness of the modeled structure. Applied loadings and self weight calculations are used to account for the mass of the structure.

For the Service Building, the seismic mass chosen to use in the mass and stiffness model generation for Sargent and Lundy's soil structure interaction analysis and the floor response spectra generation was comprised of the following:

100% of the structure self weight load +
100% of the structure dead load +
100% of the utility load (which is actually 25% Live Load per ASCE 43-05)

This combination was chosen to best represent the mass that may be present during an actual seismic event. It also was based upon experience and indicative of the combinations used on past projects for such purposes.

Due to the complexity of the preliminary RISA-3D model, for the purpose of a soil structure interaction (SSI) dynamic analysis, a condensed model (stick model) is prepared

that simplifies certain parts of the building, while still capturing the building's primary dynamic characteristics. This condensed model is intended only to approximate the lateral load resisting system and tributary masses that contribute to the fundamental mode shape.

Generating Mass per Elevation:

The RISA-3D analysis model is used to generate the seismic mass per elevation. Since the RISA- 3D model is a stiffness model based on geometry and loadings, the analysis simply resolves these forces by statics to the corresponding support joints based on its geometry and stiffness. The procedure for the generation of mass is as follows:

• All the nodes of the analysis model for the elevations of interest are assigned as fixed in order to generate a reaction at these locations.
• The model is analyzed statically for the seismic mass load combination.
• The reactions at each joint are reported by RISA-3D. Joint reactions and joint information are copied into an Excel file.
• Using Excel, joints are sorted by elevations and reactions are summed by elevation to obtain the mass at that floor level.

Masses were lumped at the ground floor (base El. 34.33'), first elevated floor (El. 50.33'), and the roof (El. 67.0').

Determining Stiffness between Mass Elevations:

The RISA-3D analysis model is used to calculate the stiffness of the structure in two orthogonal directions. This involves determining a 2D stiffness from a 3D model. In order to accomplish this, translation transverse to the direction of calculated stiffness is restrained in the 3D model. Once these restraints are assigned the stiffness can be determined by applying a known force at each elevation matching the mass summation elevations and determining the corresponding deflections. This is done from the lowest elevation to the upper most elevation. As the analysis progresses upward, rollers are applied at the lower mass elevation to prevent their translations, but not rotations. This allows for moment transfer through the lower half of the structure. RISA- 3D performs the analysis and reports the deflections. Once this is completed for every section between mass elevations, then the generated stick model is compared against the full analysis model for accuracy.

The specific procedure used is as follows:
• All nodes at elevation 34.33' are fixed.
• Diaphragms are assigned at the mass elevations.
• All nodes are restrained in the direction perpendicular to the loading.
• Loads are applied at elevation 50'-4".
• Deflections are recorded at elevation 50' - 4".
• Nodes at elevation 50' - 4" are restrained from translation in the plane of the floor.

- Loads are applied at elevation 67' - 0".
- Deflections are recorded at elevation 67' - 0".

Verification of Calculated Stiffness:

To check that the stiffness calculation is correct for the 2D model accurately representing the 3D model, a mass and stiffness model is created using elements between each mass node. The 2D model is modeled with three nodes and two mass-less elements. The 2D model is loaded at the top node (elevation 67' - 0") in the North-South and East-West direction. The 3D model is loaded with the same magnitude as the stick and in the same directions. An analysis was performed in each direction and the deflections were deemed comparable, therefore, the mass and stiffness model was an adequate representation of the Service Building in a SSI analysis.

Seismic Analysis for Stability Calculation:

A 2D stick model was used for seismic analysis for the stability calculation. The amplified site-specific SSE motions accounting for the effect of nearby Reactor and Control Buildings were used to run a response spectrum analysis on the stick model and determine the superstructure unscaled base shear in both orthogonal directions. The basemat mass was accelerated at the ZPA of the amplified curves and then added to the RSA unscaled base shear, yielding the total seismic base shear for each direction.

2) *RSA details:*
   a) *Modal combination method*
   b) *Combination of 3 directional responses*

The modal combination used for the RSA was the Complete Quadratic Combination (CQC). The three directional responses were combined using the 100-40-40 rule as outlined in Regulatory Guide 1.92, Revision 2.

3) *Attachment 01 sheets 4 and 5, what are modes 5 and 6 with frequency > $3.0x10^8$ Hz?*

Since these stick models represented stiffness in only one orthogonal direction, the model was fully restrained in the direction perpendicular to the stiffness. These modes represent rigid body motion in the restrained direction.

Section 5.14 of the calculation will be revised to include the above clarifications.

For COLA revisions as a result of this part of the response, see mark-up to COLA Sections 3.7.2.8 and 3.7.3.16 in Enclosure 1.

g. *Revise the Control Building Annex stability evaluation to use ASCE 7-05 instead of ASCE 7-88. Check for two cases, (1) with live load for both the stabilizing force and the driving force and (2) with no live load for either the stabilizing force or the driving force (Audit Action Item 3.8-42, Punch List Item 101)*

The Control Building Annex (CBA) stability evaluation calculation has been revised as requested. There is no change in the reported stability factors of safety reported in COLA Table 3H.6-14 due to the following:

- Wind loading per ASCE 7-88 is more critical than wind loading per ASCE 7-05
- Factors of safety for sliding and overturning when considering no live load are equal or higher than those with live load consideration.

h. *Staff requests additional description of foundations in FSAR 3.8.5. Applicant will provide a brief description of the foundations, foundation analysis, and differential settlement determination, including consideration of construction sequence (Punch List Item 108)*

See new COLA Sections 3.8.5.8 and 3.8.5.9 provided in Enclosure 1 for description of foundations for Diesel Generator Fuel Oil Tunnels (DGFOT) and Category I site-specific structures, respectively.

Enclosure 1

COLA MARK-UPS

These COLA Part 2, Tier 2 mark-ups are based on COLA Revision 5 and subsequent mark-ups provided in RAI responses submitted through March 25, 2011.

3.7.2.8 Interaction of Non-Seismic Category I Structures, Systems and Components with Seismic Category I Structures, Systems and Components

The Category I structures and their physical proximity to nearby non-Category I structures are shown in Figure 3.7-40. None of the non-Category I structures proposed as part of STP Units 3 and 4 is intended to meet Criterion (2) of DCD Section 3.7.2.8. Rather, for each non-Category I structure, either: (1) it is determined that the collapse of the non-Category I structure will not cause the non-Category I structure to strike a Category I structure; or (2) the non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions in a manner such that the margin of safety of the structure is equivalent to that of Seismic Category I structures. Non-Category I structures that can interact with Seismic Category I structures include the Turbine Building (TB), Radwaste Building (RWB), Service Building (SB), Control Building Annex (CBA) and the stack on the Reactor Building roof. ~~Table 3H.6-14 provides sliding and overturning factors of safety under site-specific SSE for TB, RWB, SB, and CBA.~~

The seismic input motions for the II/I design of the five non-seismic category I structures noted above are described in the following:

- TB: 0.3g Regulatory Guide 1.60 spectra.
- RWB: as described in Sections 3.7.3.16 and 3H.3.5.3 and shown in Figures 3.7-~~40~~41 through 3.7-~~42~~43.
- SB: as described in Section 3.7.3.16.~~0.3g Regulatory Guide 1.60 spectra.~~
- CBA: as described in Section 3.7.3.16 and shown in Figures 3.7-38 and 3.7-39.

Stack on the Reactor Building roof: seismic loading at its location, resulting from the SSE analysis of the Reactor Building.

The seismic input motions for II/I stability evaluations of TB, RWB, SB, and CBA are described in the following:

- TB: site-specific SSE
- RWB: as described in Sections 3.7.3.16 and 3H.3.5.3 and shown in Figures 3.7-44 through 3.7-46
- SB: as described in Section 3.7.3.16
- CBA: as described in Section 3.7.3.16

Sliding and overturning stability evaluations of TB, RWB, SB, and CBA are performed in accordance with the methodology outlined in Figure 3H3-52.

Seismic demands along each orthogonal direction for stability evaluation of TB, RWB, and SB are determined using response spectrum analysis of a fixed base stick model representing each of these structures. The input motions for these response spectrum analyses are as described above. The base shears and moments from these response spectrum analyses are adjusted manually to account for the additional shears and moments due to basemat excitation which are calculated considering zero period acceleration (ZPA) of the input motions. The three orthogonal seismic

demands of each structure are combined using the 100%-40%-40% rule as outlined in Regulatory Guide 1.92, Revision 2.

Seismic demands along each orthogonal direction for stability evaluation of the CBA are calculated using manual calculation where the CBA is idealized as a single degree of freedom structure. The three orthogonal seismic demands of each structure are combined using the 100%-40%-40% rule as outlined in Regulatory Guide 1.92, Revision 2.

Table 3H.6-14 provides sliding and overturning factors of safety under site-specific SSE for TB, RWB, SB, and CBA.

3.7.3.16 Analysis Procedure for Non-Seismic Structures in Lieu of Dynamic Analysis

For the Control Building Annex (CBA) II/I design, the SSE input at the foundation level (Figures 3.7-38 and 3.7-39) is the envelope of 0.3g RG 1.60 response spectra and the induced acceleration response spectra due to site specific SSE that is determined from an SSI analysis which accounts for the impact of the nearby Control Building (CB). In this SSI analysis, five interaction nodes at the depth corresponding to the bottom elevation of the CBA foundation are added to the three dimensional SSI model of the CB. These five interaction nodes correspond to the four corners and the center of the CBA foundation. The average response of these five interaction nodes is enveloped with the 0.3g RG 1.60 spectra to determine the SSE input at the CBA foundation level.

For the stability evaluation of the CBA, the SSE input is the envelope of the average response of the five interaction nodes from the SSI analysis described above and the site specific SSE.

For the Radwaste Building (RWB) II/I design, the SSE input (see Figures 3.7-41 through 3.7-43) ~~at the foundation level~~ is the envelope of 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum due to site-specific SSE that is determined from an SSI analysis which accounts for the impact of the nearby Reactor Building (RB). In this SSI analysis, five interaction nodes at the ~~depth~~ground surface ~~corresponding to the bottom elevation of the RWB foundation~~ are added to the three dimensional SSI model of the RB. These five interaction nodes correspond to the four corners and the center of the RWB foundation. The average response of these five interaction nodes is enveloped with the 0.3g RG 1.60 spectra to determine the SSE input at the foundation level.

For the stability evaluation of the RWB, the SSE input (see Figures 3.7-44 through 3.7-46) is the envelope of the average response of the five interaction nodes from the SSI analysis described above and the site specific SSE.

For the Service Building (SB) II/I design, the SSE input is the envelope of 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum due to site-specific SSE that is determined from an SSI analysis which accounts for the impact of the nearby CB Building. In this SSI analysis, five interaction nodes at the ground surface are added to the three dimensional SSI model of the CB. These five interaction nodes correspond to the four corners and the center of the SB foundation. The average response of these five interaction nodes is enveloped with the 0.3g RG 1.60 spectra to determine the SSE input at the foundation level.

For the stability evaluation of the SB, the SSE input is the envelope of the average response of the five interaction nodes from the SSI analysis described above and the site specific SSE.

3.8.5.8.1 Description of Foundations for DGFOT

Diesel Generator Fuel Oil Tunnels (DGFOT) foundation is a 2 ft thick reinforced concrete basemat placed over two feet thick lean concrete mud mat. The foundation analysis and design is performed using a three dimensional finite element analysis (FEA). The flexibility of the basemat and the supporting soil is accounted for through use of foundation soil springs. For additional analysis and design details, see Section 3H.7.

Seismic gaps between the DGFOT and adjoining Reactor Building (RB) and Diesel Generator Fuel Oil Storage Vaults (DGFOSV) as well as the differential movements for design commodities communicating between the DGFOT and the adjoining RB and DGFOSV are determined considering settlement and tilts obtained from time rate of settlement analysis accounting for construction sequence, seismic movements from seismic analysis, and translations and/or rotations from sliding and overturning stability evaluations.

3.8.5.9 Description of Foundations for Category I Site-Specific Structures

3.8.5.9.1 UHS/RSW Pump House

Ultimate Heat Sink (UHS)/Reactor Service Water (RSW) Pump House foundation is a 10 ft thick reinforced concrete basemat placed over two feet thick lean concrete mud mat. The foundation analysis and design is performed using a three dimensional finite element analysis (FEA). The flexibility of the basemat and the supporting soil is accounted for through use of foundation soil springs. For additional analysis and design details, see Section 3H.6.

Seismic gaps between the RSW Pump House and the adjoining RSW Piping Tunnels as well as the differential movements for design of commodities communicating between the RSW Pump House and RSW Piping Tunnels are determined considering settlement and tilts obtained from time rate of settlement analysis accounting for construction sequence, seismic movements from seismic analysis, and translations and/or rotations from sliding and overturning stability evaluations.

3.8.5.9.2 Reactor Service Water (RSW) Piping Tunnels

RSW Piping Tunnels foundation is a three ft thick reinforced concrete basemat placed over 2 ft thick lean concrete mud mat. The foundation analysis and design is performed using conservative manual calculations as described in Section 3H.6.6.2.2.

Seismic gaps between the RSW Piping Tunnels and the adjoining Control Building (CB) and RSW Pump House as well as the differential movements for design of commodities communicating between the RSW Piping Tunnels and the adjoining CB and RSW Pump House are determined considering settlement and tilts obtained from time rate of settlement analysis accounting for construction sequence, seismic movements from seismic analysis, and translations and/or rotations from sliding and overturning stability evaluations.

3.8.5.9.3 Diesel Generator Fuel Oil Storage Vaults (DGFOSV)

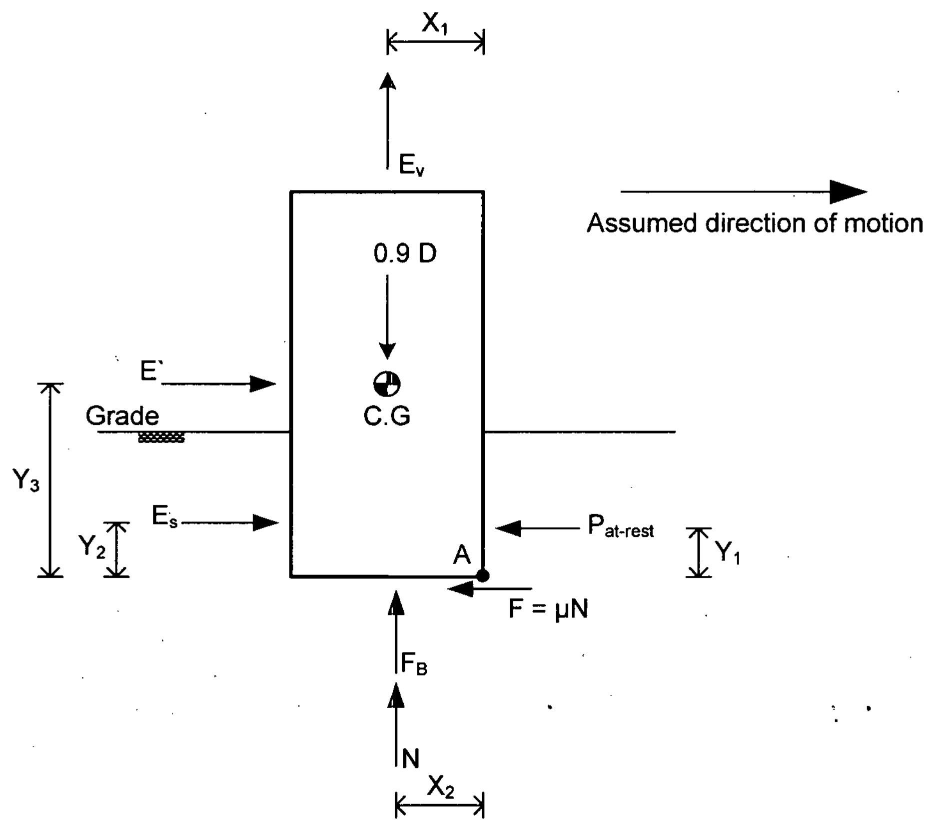
DGFOSV foundation is a 6 ft thick reinforced concrete basemat placed over 2 ft thick lean concrete mud mat. The foundation analysis and design is performed using a three dimensional finite element analysis (FEA). The flexibility of the basemat and the supporting soil is accounted for through use of foundation soil springs. For additional analysis and design details, see Section 3H.6.7.

Seismic gaps between the DGFOSV and the adjoining DGFOT as well as the differential movements for design commodities communicating between the DGFOSV and DGFOT are determined considering settlement and tilts obtained from time rate of settlement analysis accounting for construction sequence, seismic movements from seismic analysis, and translations and/or rotations from sliding and overturning stability evaluations.

Table 3H.6-14: Calculated Overturning and Sliding Factors of Safety Under Site-Specific SSE and Flotation Factors of Safety for TB, SB, RWB and CBA

Structure	Calculated Factor of Safety			Minimum Required Factor of Safety	Coefficient of Friction for Sliding Evaluation
	Overturning	Sliding	Flotation		
Turbine Building (TB)	2.18	1.11	1.46	1.1	0.30 (dynamic)
Service Building (SB)	~~2.65~~ 2.11	~~1.81~~ 1.11	1.40	1.1	0.39 (dynamic)
Radwaste Building (RWB)	~~4.23~~ 3.67	~~1.92~~ 1.75	1.51	1.1	0.39 (dynamic)
Control Building Annex (CBA)	2.03	1.16	1.18	1.1	0.58 (static)

Figure 3H.6-137: Formulations Used for Calculation of Factors of Safety Against Sliding and Overturning for Category I Site-Specific Structures



Factors of Safety against Sliding and Overturning about point A are calculated as follows:

$$SF_{sliding} = \frac{P_{at\_rest} + F}{E_s + E`}$$

$$SF_{OT\_A} = \frac{(P_{at\_rest})(Y_1) + (0.9D)(X_1)}{(F_B)(X_2) + (E_s)(Y_2) + (E`)(Y_3) + (E_v)(X_1)}$$

Where:

$SF_{sliding}$ = Safety factor against sliding

SF_{OT_A} = Safety factor against overturning about "A"

D = Dead load

P_{at_rest} = Total at-rest soil pressure (see Figures 3H.6-48 through 3H.6-50)

$F = \mu N$ = friction force and μ is the coefficient of friction

E_s = Static and dynamic soil pressure (see Figures 3H.6-45 through 3H.6-47)

E` = Self weight excitation in the horizontal direction

E_v = Self weight excitation in the vertical direction

F_B = Buoyancy force

N = Vertical reaction $= 0.9D - F_B - E_v$

Note: If passive pressure is utilized, $P_{passive}$ should be used instead of $P_{at\text{-}rest}$.

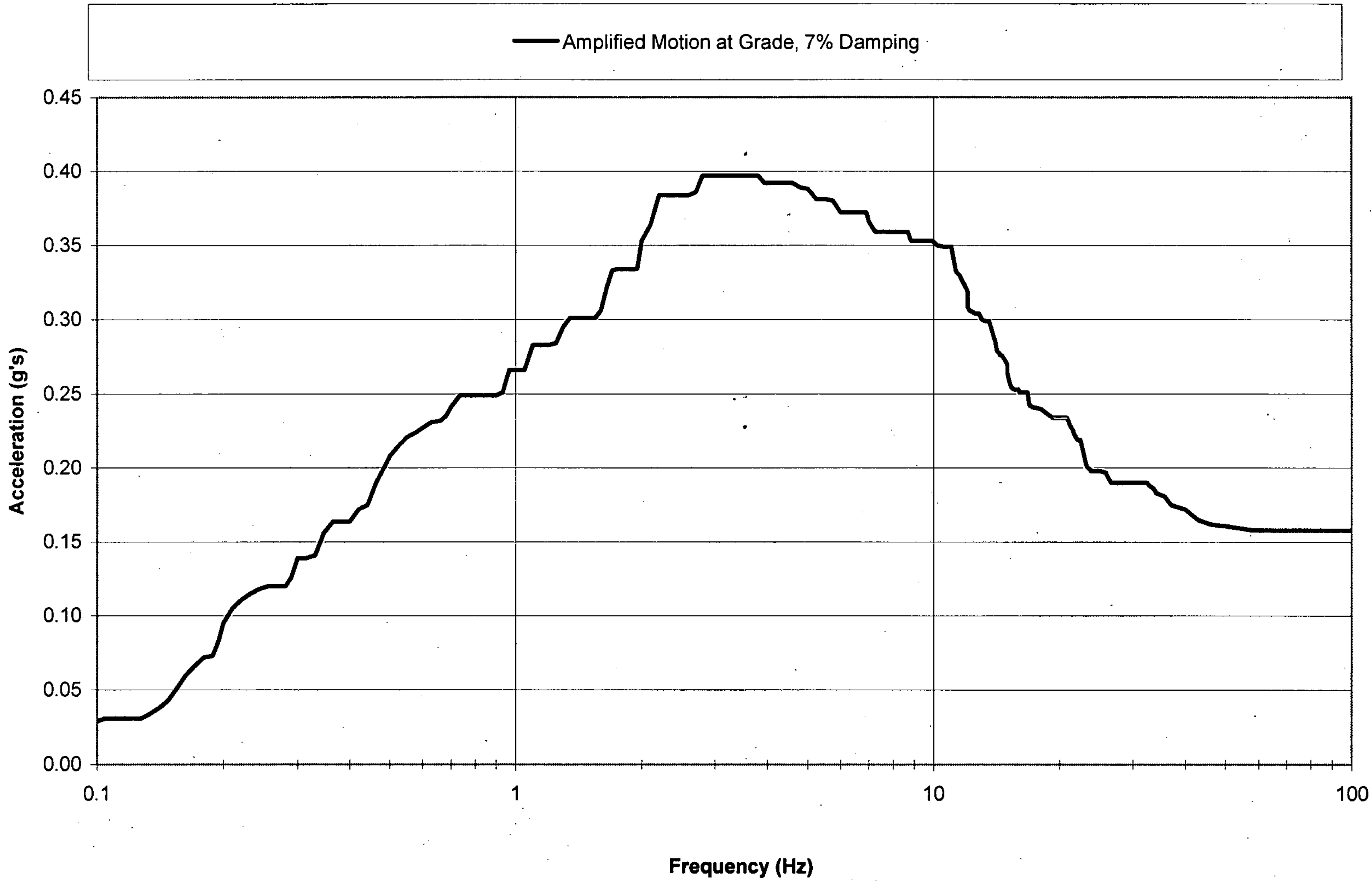


Figure 3.7-44 Radwaste Building East-West Input Motion for Stability Evaluations (7% Damping)

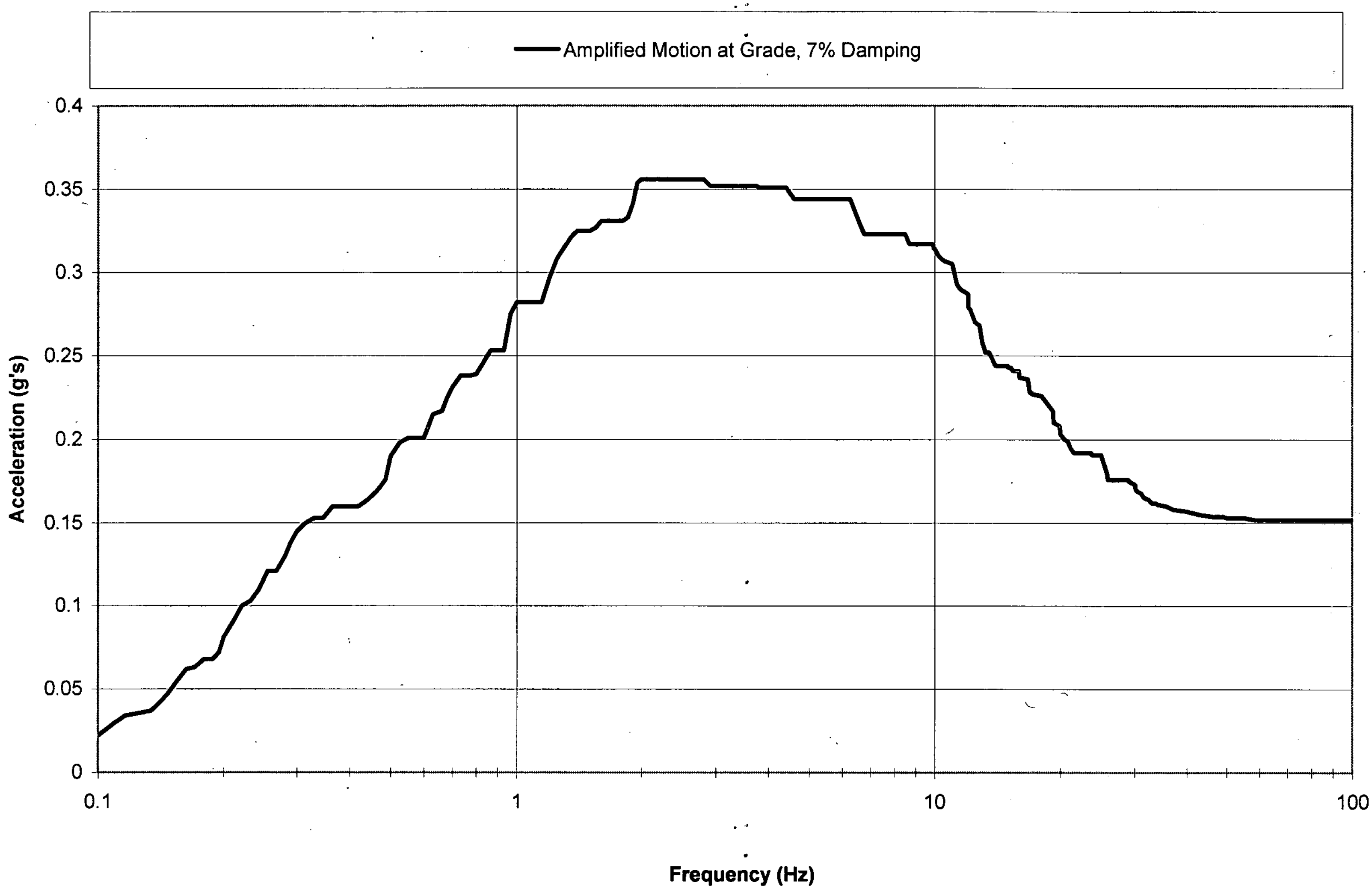


Figure 3.7-45 Radwaste Building North-South Input Motion for Stability Evaluations (7% Damping)

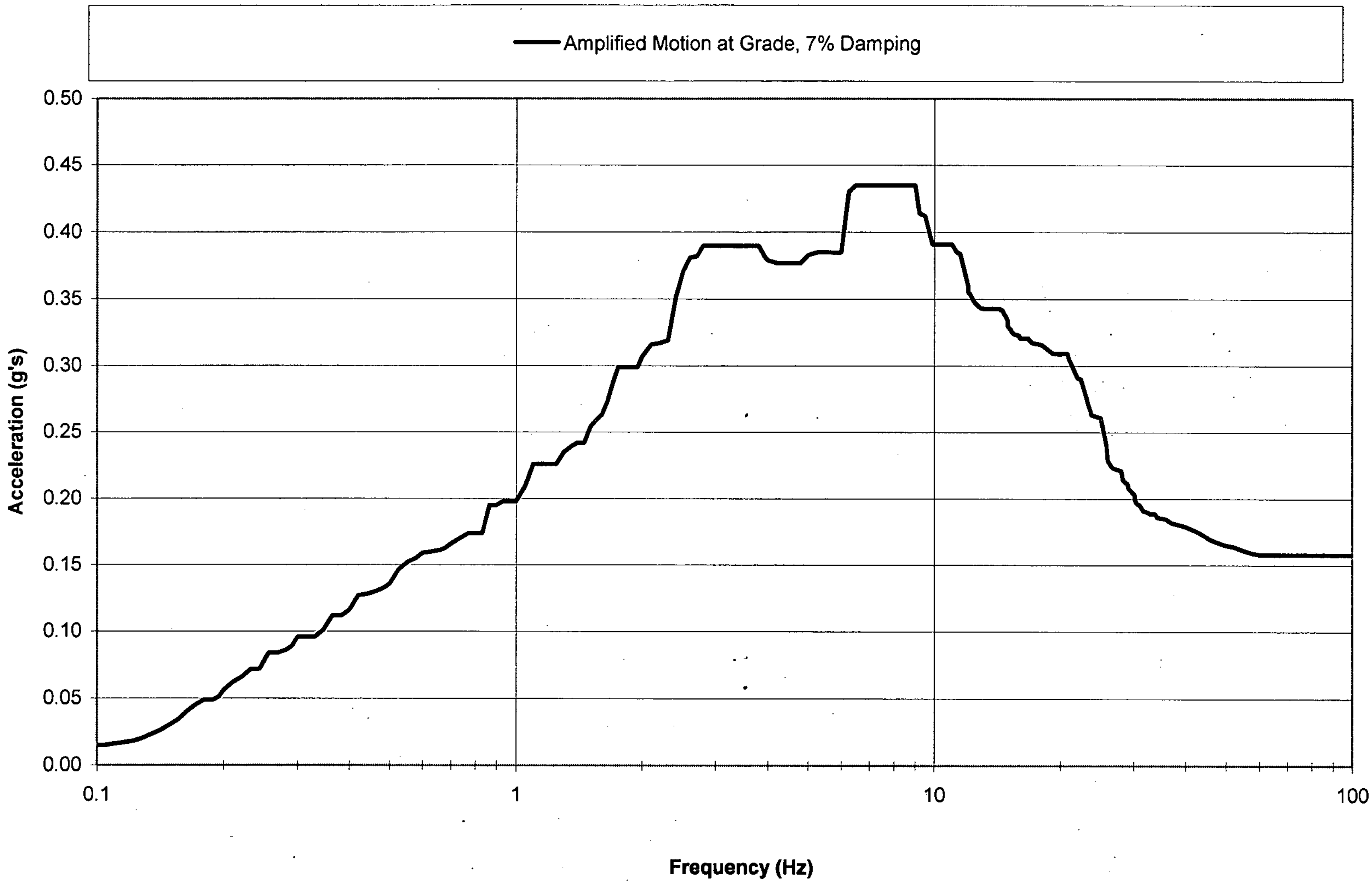


Figure 3.7-46 Radwaste Building Vertical Input Motion for Stability Evaluations (7% Damping)

RAI 03.08.04-18, Supplement 3

QUESTION:

Follow-up to Question 03.08.04-2 (RAI 2964)

The applicant's response to Question 03.08.04-2 states that the Radwaste Building (RWB) will be designed in accordance with the requirements of RG 1.143, Revision 2. The applicant also discussed the design criteria for this building for seismic category II/I evaluation. In order for the staff to conclude that the Radwaste Building design meets the requirements of RG 1.143, and also meets the requirement in ABWR DCD Section 3.7.2.8, item (3), the FSAR needs to include sufficient design information for the building to demonstrate that the design meets the pertinent design criteria. Guidance provided in SRP Section 3.8.4 may be used for providing such information. Therefore, the applicant is requested to provide design information for the RWB in the FSAR that includes more detailed description of the structure; applicable codes, standards and specifications; loads and load combinations including live loads, seismic loads, thermal loads, flood loads, tornado loads, lateral soil pressure, etc.; design and analysis procedures; structural acceptance criteria; materials and quality control; design of critical sections, stability evaluation, etc.

SUPPLEMENTAL RESPONSE:

The Revision 1, Supplement 2 response to this RAI was submitted with Nuclear Innovation North America (NINA) letter U7-C-NINA-NRC-110043, dated March 15, 2011. This supplement provides the response to the following action item discussed in the NRC audit performed during the week of May 23, 2011.

For Radwaste Building, revise the COLA for the following (Audit Action Item 3.8-37, Punch List Item 92):

1) *Add clarification in Section 3H.3.5.1, 2 and 3 for seismic analysis models (stick model for stability, FEM model for design and II/I) [Clarification Issue 13.1]*

   For the requested clarifications, see revised COLA Sections 3H.3.5.1 through 3H.3.5.3 provided in Enclosure 1.

   Note: COLA mark-up for Section 3H.3.5.3 refers to Sections 3.7.2.8 and 3.7.3.16. Revised COLA Sections 3.7.2.8 and 3.7.3.16 are provided in RAI 03.07.02-13 S3 which is being submitted concurrently with this response.

2) *Delete vertical accelerations from Table 3H.3-1 [Clarification Issue 13.1]*

   See Enclosure 1 for revised COLA Table 3H.3-1.

3) *Revise Section 3H.3.4.3.4.2 to include N690 provision for shear allowable [Clarification Issue 13.3]*

   For inclusion of N690 code provisions for allowable shear stress, see revised COLA Sections 3H.3.4.3.4.2 and 3H.3.5.3.1.2 provided in Enclosure 1.

4) *Revise Section 3H.3.4.2.1 to state that bearing pressure capacity is determined using methodology described in COLA Section 2.5S.4 [Clarification Issue 13.4]*

   See Enclosure 1 for revised COLA Section 3H.3.4.2.1.

Enclosure 1

COLA MARK-UPS

These COLA Part 2, Tier 2 mark-ups are based on COLA Revision 5 and subsequent mark-ups provided in RAI responses submitted through March 25, 2011.

3H.3.4.2.1 Soil Parameters

- Poisson's ratio (above groundwater).. 0.42
- Poisson's ratio (below groundwater) 0.47
- Unit Weight (moist)..120 pcf
- Unit Weight (saturated) ....140 pcf
- Liquefaction potential ....None
- Static Soil Bearing Pressure (plus weight of 2 ft of fill concrete):................9.8 ksf
- Ultimate Static Soil Bearing Capacity..91.1 ksf
- Static Soil Bearing Capacity Factor of Safety.. ≥ 9.3
- Dynamic Soil Bearing Pressure:..11.0 ksf
- Ultimate Dynamic Soil Bearing Capacity..71.4 ksf
- Dynamic Soil Bearing Capacity Factor of Safety.. ≥ 6.5

The soil bearing pressure capacities noted above are determined using the methodology described in Section 2.5S.4.

3H.3.4.3.4.2 Structural Steel Load Combinations

S = D + L + F + H + Ro + To

1.33S = D + L + F + H + Ro + Tb

1.33S = D + L + F + H + Ro + To + W

1.33S = D + L + F + H' + Ro + To + Eo

1.33S = D + L + F + H + Ro + To + FL

1.6S (Note 1) = D + L + F + H + Ro + To + Wt

For the computation of global seismic loads, the live load is limited to the expected live load present during normal plant operation which is defined as 25% of the normal floor and roof live loads. However, design of local elements such as beams and slabs is based on consideration of full normal live load.

Note 1: The stress limit coefficient in shear shall not exceed 1.4 in members and bolts.

3H.3.5.1 Seismic Analysis

Two types of seismic analyses are performed for the RWB. The analysis and design of the RWB as well as the II/I design is performed using response spectrum analysis of a SAP2000 3D finite element model described in Section 3H.3.5.2. The II/I stability evaluation of the RWB is performed using the base shears and moments

obtained from response spectrum analysis of a fixed base stick model described below. This fixed base stick model is also used for obtaining the seismic in-plane shears and moments of the exterior walls reported in Table 3H.3-1 and the structural frequencies reported in Table 3H.3-2.

~~The seismic analysis of the RWB is performed using a fixed base stick model.~~ In the fixed base stick model, ~~T~~the structure is represented by a lumped-mass model consisting of structural masses lumped at selected nodes which are connected by massless elements representing the stiffness properties of the shear walls between the nodes. The building masses are lumped at elevations where the building weights are concentrated such as the floors and roof.

For modeling reinforced concrete shear wall elements, the shear walls in each particular vibration direction are identified. The stiffness of a shear wall along its length consists of a combination of its shear stiffness and its flexural stiffness, both of which are calculated individually and combined to obtain the stiffness of the wall.

~~The input motion of the seismic analysis is the Regulatory Guide 1.60 response spectra for 0.15g.~~

~~The RWB seismic design loads are shown in Table 3H.3-1. The RWB structural frequencies are shown in Table 3H.3-2.~~

3H.3.5.2 Analysis and Design

The analysis and design of the RWB is performed using a SAP2000 3D finite element model with shell and frame elements, as shown in Figures 3H.3-5 through 3H.3-7. The seismic loads are obtained from response spectrum analysis of this model. The input motion for this response spectrum analysis is the Regulatory Guide 1.60 response spectra for 0.15g.

Per Table 1 of RG 1.143 Revision 2, all concrete and steel designs are in accordance with the ACI 349-97 and ANSI/AISC N690, 1984 code requirements, respectively. ~~Also, for II/I design, the structure is conservatively designed to remain elastic.~~

The forces and moments at critical locations in the Radwaste Building along with the provided longitudinal and transverse reinforcement are included in Table 3H.3-3 for the exterior walls and Table 3H.3-4 for the basemat, roof slab, and operating floor (elevation 35'-0") slab. Figures 3H.3-8 through 3H.3-27 show the location of the reinforcement zones listed in Table 3H.3-3 for the exterior walls. Figures 3H.3-28 through 3H.3-42 show the location of the reinforcement zones listed in Table 3H.3-4 for the basemat, roof slab, and operating floor slab.

The structural steel member sizes, critical forces, safety margins, and governing load combinations for the operating floor beams, roof truss members, and roof purlins are shown in Table 3H.3-5. The layout of the operating floor steel beams is shown in Figures 3H.3-43 through 3H.3-46. The layout of the roof truss members and roof purlins are shown in Figure 3H.3-47. The typical east-west spanning truss

and typical north-south spanning truss are shown in Figures 3H.3-48 and 3H.3-49, respectively.

3H.3.5.3 Seismic II/I Evaluation

The seismic II/I evaluation for the RWB is performed to ensure that the RWB will not collapse on the nearby Category I structures. The analysis and design for II/I is performed using a SAP2000 3D finite element model with shell and frame elements, as shown in Figures 3H.3-5 through 3H.3-7. The seismic loads are obtained from response spectrum analysis of this model. ~~The structure is conservatively designed to remain elastic for this evaluation.~~ The earthquake input used at the foundation level is the envelope of 0.3g RG 1.60 response spectrum and the induced acceleration response spectrum due to site-specific SSE that is determined from an SSI analysis which accounts for the impact of the nearby Reactor Building (RB). In this SSI analysis, five interaction nodes at ~~the depth corresponding to the bottom elevation of the RWB foundation~~ground surface are added to the three dimensional SSI model of the RB. These five interaction nodes correspond to the four corners and the center of the RWB foundation. The average response of these five interaction nodes is enveloped with the 0.3g RG 1.60 spectra to determine the SSE input at the foundation level. The structure is conservatively designed to remain elastic for this evaluation.

For tornado parameters, including the missiles, the same parameters as those defined in DCD Tier 1 Table 5.0 are used. For flood, the extreme flood level of 40 ft (12.2 m) MSL is used, which is caused by the Main Cooling Reservoir dike breach. The evaluation requirements for this flood, including hydrodynamic and flooding debris loading, are included in Section 3.4.2.

The II/I stability evaluations for sliding and overturning are performed using the seismic input motion described in Sections 3.7.2.8 and 3.7.3.16 ~~sitespecific SSE~~ and other site-specific parameters such as soil properties. The seismic demands for II/I stability evaluation are determined by response spectrum analysis of the fixed base stick model described in Section 3H.3.5.1. Figure 3H.3-52 outlines the methodology followed for the seismic II/I stability evaluation of the RWB.

3H.3.5.3.1.2 Structural Steel Load Combinations

1.6S (Note 1) = D + L + F + H' + Ro + To + E'

For the computation of global seismic loads, the live load is limited to the expected live load present during normal plant operation which is defined as 25% of the normal floor and roof live loads.

Note 1: The stress limit coefficient in shear shall not exceed 1.4 in members and bolts.

Table 3H.3-1 Radwaste Building Design Seismic Loads

Wall	Elevation (ft)	In-Plane Forces (1) 1/2 SSE (0.15g) (kips)	In-Plane Moments(1) 1/2 SSE (0.15g) (kips-ft)
North Wall	95'-0"	5963	0
	35'-0"	4133	351845
	(-)11'-0"	9328	770605
South Wall	95'-0"	5351	0
	35'-0"	2888	315719
	(-)11'-0"	7186	635566
East Wall	95'-0"	4555	0
	35'-0"	3276	268725
	(-)11'-0"	7282	595912
West Wall	95'-0"	5481	0
	35'-0"	4362	323390
	(-)11'-0"	9125	732302

Notes:
(1) The forces and moments reported are the maximum calculated for all time steps. Therefore, the summation of the forces at Elevation 35'-0" and Elevation 95'-0" is not equal to the force at Elevation (-)11'-0".

RAI 03.08.04-33 Supplement 1

QUESTION:

1. In FSAR Section 3.8, page 3.8-1, the applicant references the departure STD DEP 1.8-1, "Tier 2* Codes, Standards, and Regulatory Guide Edition Changes." One of the changes included in this departure updates Tier 2 to refer to the 1997 edition of ACI 349 in place of the 1980 edition of the same building code for concrete structures. In the ABWR design certification (NUREG-1503, page 3-53), the staff had evaluated only the use of 1980 edition of ACI 349. Therefore, the applicant is requested to provide a detailed comparison of the differences between these two editions of the code as they apply to the ABWR standard design, and provide justifications for any differences in order for the staff to evaluate the acceptability of the 1997 edition of ACI 349.

2. FSAR Section 3H.6.4.1 references ANSI/AISC N690 specification for design, fabrication, and erection of site-specific seismic category I steel structures. The applicant did not specify in this section which version of the specification is used. It appears that the applicant uses the 1984 edition of the specification referenced in ABWR DCD Table 1.8-21, which the applicant incorporated by reference. However, according to SRP acceptance criteria 3.8.4.II.5, ANSI/AISC N690-1994 including Supplement 2 (2004) has been accepted by the staff for design, fabrication, and erection of safety-related steel structures. According to the guidance provided in RG 1.206, Section C.I.1.9.2, the applicant should use the current SRP for structures outside the scope of the ABWR DCD, or provide justification for not doing so. Therefore, the applicant is requested to provide a detailed comparison of the differences between the 1984 (or whatever edition is used by the applicant) and the 1994 editions of the specification as they apply to the site-specific seismic category I structures at STP site. Also, provide the justification(s) for any differences in order for the staff to evaluate the acceptability of the 1984 edition of the specification.

3. Furthermore, the staff observed that Table 1.8-21 in FSAR Tier 2, Section 1.8, references ASME Code, Section III, Division 2, Edition 2001 with 2003 addenda, and identifies certain limitations. The ABWR DCD specifies the use of ASME code version 1989. In the ABWR FSER, p. 3-49, the NRC has accepted the 1989 Edition of the ASME Code, Section III, Division 2. Therefore, the applicant is requested to provide a detailed comparison of the differences between these two editions of the code as they apply to the design and analysis of safety-related ABWR standard plant structures, and provide justification(s) for any differences in order for the staff to evaluate the acceptability of the ASME Code, Section III, Division 2 Edition 2001 with 2003 addenda. The applicant is also requested to explain how use of the Edition of the ASME Code proposed by the applicant meets the provisions of NCA-1140, "Use of Code Editions, Addenda, and Cases."

The staff needs the above information to conclude that the applicant used acceptable codes and standards for all seismic category I structures, and any deviations are appropriately addressed.

SUPPLEMENTAL RESPONSE:

The original response to this RAI was submitted with STP Letter U7-C-STP-NRC-100208, dated September 15, 2010. This supplemental response addresses the following Clarification Issue Number 23, as provided by NRC (Punch List Item Number 107):

"In its response to RAI 03.08.04-33 with letter U7-C-STP-NRC-100208 Attachment 21 Page 3 of 7, the applicant stated that:

> *"Appendix B - Steel Embedments, includes changes in ACI 349-97 based on later research. The changes in Appendix B are for the local design of embedment plates and do not affect the design of the major concrete elements. Additionally, the supplemental requirements defined in the Staff Positions in DCD Table 3.8 -10 will have a larger impact on the embedment design."*

It appears that the applicant is comparing the provisions of ACI 349-97, Appendix B, with those of the ABWR DCD Table 3.8-10. However, ACI 349-97, Appendix B was not endorsed by the NRC.

In its evaluation the staff noted that according to FSAR, Rev. 04, Table 1.8-20, RG 1.142 is changed to Revision 2, Nov 2001, which endorses ACI 349-97 with the exception of Appendix B (Anchoring to Concrete): "This regulatory guide delineates the extent to which ACI 349-97, except for Appendix B, "Steel Embedments," is acceptable to the NRC staff. In a separate action, the staff intends to endorse Appendix B of ACI 349-97 in a regulatory guide that is being developed to address anchoring components and structural supports in concrete."

SRP 3.8.3, II.2, Rev 2, March 2007 lists under applicable codes ACI 349 as supplemented with additional guidance by RG 1.142 and 1.199.

SRP 3.8.4, II.4 (A), Rev 2, March 2007 lists as acceptance criteria: "For concrete structures, the procedures are in accordance with ACI 349, as supplemented by RG 1.142. The design and analysis of anchors (steel embedments) used for component and structural supports on concrete structures are acceptable if found in accordance with Appendix B to ACI 349, as supplemented by RG 1.199."

RG 1.199, Nov. 2003, under C. Regulatory Position, states: "1. The procedures and standards of Appendix B to ACI 349-01 are acceptable to the NRC staff as described and supplemented below. The recommendations are applicable to the types of anchors discussed in Section B.1, "Definitions," and B.2, "Scope," of Appendix B to ACI 349-01."

Since RG 1.199 is not listed in FSAR, Rev. 4, Table 1.8-20 as an applicable guidance to STP

3&4, the applicant is requested to clarify which provisions are being used for steel embedments for both the standard plant structures and the site-specific structures."

COLA Part 2, Tier 2 Table 1.9S-1 already commits to Regulatory Guide 1.199, Rev.0, "Anchoring Components and Structural Supports in Concrete", for design of site-specific systems, structures, and components (SSCs). As requested by NRC, the COLA mark-up provided in the Enclosure to this response revises Tier 2 Table 1.8-20 associated with Departure STD DEP 1.8-1 to commit to Regulatory Guide 1.199 for the design of the Standard Plant SSCs also. With this commitment the DCD Tier 2 Table 3.8-10 is no longer required and is, therefore, deleted.

COLA will be revised as shown in Enclosure 1.

Enclosure 1

COLA MARK-UPS

These COLA Part 2 mark-ups are based on COLA Revision 5 and subsequent mark-ups provided in RAI responses submitted through March 25, 2011.

1.8 Conformance with Standard Review Plan and Applicability of Codes and Standards

Revisions have been made to Table 1.8-20 and are summarized here. The STP 3 & 4 FSAR conforms with the following revisions of Regulatory Guides (RGs).

RG 1.75 Rev. 3

RG 1.82, Rev. 3

RG 1.84, Rev. 33

RG 1.136, Rev. 3

RG 1.142, Rev. 2

RG 1.143, Rev. 2

RG 1.153, Rev. 1

RG 1.199, Rev. 0

RG 1.85 has been deleted (withdrawn).

Table 1.8-20 NRC Regulatory Guides Applicable to ABWR (Continued)

[1.153 Criteria for Power, Instrumentation, and Control Portions of Safety Systems	~~*0 12/85*~~ **1 6/96**	*Yes](4)*
1.199 Anchoring Components and Structural Supports in Concrete	0 11/03	Yes

3.8.4.2.1 Reactor Building

The major portion of the Reactor Building is not subjected to the abnormal and severe accident conditions associated with a containment. A listing of applicable documents follows:

(1) [*ACI 349, Code Requirements for Nuclear Safety-Related Concrete Structures ~~(as modified by Table 3.8-10).~~*]*

Table 3.8-4 Codes, Standards, Specifications, and Regulations Used in the Design and Construction of Seismic Category I Internal Structures of the Containment

Specification Reference Number	Specification or Standard Designation	Title
13	[*ACI 349*	*Code Requirements for Nuclear Safety-Related Concrete Structures ~~(as modified by Table 3.8-10)~~*]*

Table 3.8-10 ~~Staff Position on Steel Embedments~~Not Used

~~[The use of Appendix B to ACI 349 for the design of steel embedments for safety-related concrete structures in ABWR is acceptable when supplemented by the following provisions.~~

~~(1) Section B.4.2 - Tension and Figures B.4.1 and B.4.2.~~

~~This section and the figures specify that the tensile strength of concrete for any anchorage can be calculated by a 45 degree failure cone theory. The staff has disseminated the German test data questioning the validity of the 45 degree failure cone theory to licensees, A/Es, bolt manufacturers, and the code committee members in its meetings with them. The data indicated that the actual failure cone was about 35 degree and the use of the 45 degree cone theory could be unconservative for anchorage design, especially for anchorage of groups of bolts. The Code Committee, having gone through some research of its own, recently agreed with the staff's position. Changes to this section are in the making by the Code Committee. In the meantime, the staff position on issues related to this Section is to ensure adoption of design approaches consistent with the test data through case by case review.~~

~~(2) Section B.5.1.1 – Tension~~

~~This section states a criterion for ductile anchors. The criterion is that the design pullout strength (force) of the concrete as determined in Section B.4.2 exceeds the minimum specified tensile strength (force) of the steel anchor,. Any anchor that meets this criterion is qualified as a ductile anchor and, thus, a low safety factor can be used. The staff believes that the criterion is deficient in two areas. One is that the design pullout strength of the concrete so calculated is usually higher than the actual strength, which has been stated in Section B.4.2 above. The other is that anchor steel characteristics are not taken into consideration. For example, the Drillco Maxi-Bolt Devices, Ltd. claims that its anchors are ductile anchors and, thus, can use a low safety factor. The strength of the Maxi-Bolt is based on the yield strength of the anchor steel, which is 723.9 MPa. The embedment length of the anchor, which is used to determine the pullout strength of the concrete, is based on the minimum specified tensile strength of the anchor steel of 861.8 MPa. The staff believes that the 19% margin (125/105) for the embedment length calculation is insufficient considering the variability of parameters affecting the concrete cone strength. The staff also questions the energy absorption capability (deformation capability after yield) of such a high strength anchor steel. Therefore, in addition to the position taken with regard to Section B.4.2 above, the staff will review vendor or manufacturer specific anchor bolt behaviors to determine the acceptable design margins between anchor bolt strengths and their corresponding pullout strengths based on concrete cones.~~

~~*Section B.5.1.1(a) - Lateral bursting concrete strength*~~

~~*This section states that the lateral bursting concrete strength is determined by the 45 degree concrete failure cone assumption. Since this assumption is wrong and likely to be replaced as stated before, the staff believes that the lateral bursting concrete strength determination is also wrong and needs to be replaced. The staff will review the anchor bolts and lateral bursting force created by the pulling of anchor bolts against test data to determine if adequate reinforcement against lateral bursting force need to be provided on a case by case basis.*~~

~~*(3) Section B.5.1.2.1 - Anchor, Studs, or Bars*~~

~~*This section states that the concrete resistance for shear can be determined by a 45 degree half-cone to the concrete free surface from the centerline of the anchor at the shearing surface. Since the 45 degree concrete failure cone for tension has been found to be incorrect, the staff believes that the use of the 45 degree half-cone for shear should be re-examined. In the meantime, the staff will review the adequacy of shear capacity calculation of concrete cones on a case by case basis with emphasis on methodology verification through vendor specific test data.*~~

~~*(4) Section B.5.1.2.2(c) - Shear Lugs*~~

~~*This section states that the concrete resistance for each shear lug in the direction of a free edge shall be determined based on the 45 degree half-cone assumption to the concrete free surface from the bearing edge of the shear lug. This is the same assumption as used in Section B.5.1.2.1 and the staff has the same comment as stated in that section. Therefore, the staff position related to the design of shear lugs is to perform case-by-case reviews. The staff review will emphasize methodology verification through sdpecific test data.*~~

~~*(5) Section B.7.2 - Alternative design requirements for expansion anchors*~~

~~*This section states that the design strength of expansion anchors shall be 0.33 times the average tension and shear test failure loads, which provides a safety factor of 3 against anchor failure. The staff position on safety factor for design against anchor failure is 4 for wedge anchors and 5 for shell anchors unless a lower safety factor can be supported by vendor specific test data.*~~

~~*(6) Anchors in tension zone of supporting concrete*~~

~~*When anchors are located within a tensile zone of supporting concrete, the anchor capacity reduction due to concrete cracking shall be accounted for in the anchor design.]**~~

Markup for COLA Part 7

2.1 Tier 1 and Tier 2* Departures from the DCD

STD DEP 1.8-1, Tier 2* Codes, Standards, and Regulatory Guide Edition Changes

Description

The American Concrete Institute code ACI 349 is updated to the 1997 edition. The ASME Section III Division 2 is updated to the 2001 edition with 2003 Addenda. These combined recognize advances in earthquake engineering and allows efficient use of modularization during construction. Note that ASME Section III Division 1 for piping is not changed from the 1989 edition. This departure also updates Tier 2 to refer to Regulatory Guides 1.136, "Materials, Construction, and Testing of Concrete Containments," Revision 3, dated 3/07, and Regulatory Guide1.142, "Safety-Related Concrete Structures for Nuclear Power Plants" to Revision 2, dated 11/01 and adds Regulatory Guide 1.199, "Anchorage Components and Structural Supports in Concrete", Rev. 0, dated 11/03. With addition of this Regulatory Guide, Table 3.8-10 is no longer required and is, therefore, deleted. Also, this departure updates Tier 2 to refer to the 2006 International Building Code (IBC), deleting the 1991 Uniform Building Code (UBC). This change incorporates the requirements of Texas building code which adopted 2006 IBC.