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June 28, 2011
U7-C-NINA-NRC-110087

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
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Rockville MD 20852-2738

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). Attachments 1 and 2 provide supplemental or revised responses to NRC staff questions included in Request for Additional Information (RAI) 03.08.04-30 and RAI 03.08.04-35 related to COLA Part 2, Tier 2, Section 3.8.

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.

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

Executed on 6/28/11

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

jep

Attachments:

RAI 03.08.04-30, Supplement 4
RAI 03.08.04-35, Revision 1

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NRO

STI 32891511

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

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RAI 03.08.04-30, Supplement 4

QUESTION:

Follow-up to Question 03.08.04-23

In response to staff question requesting additional information (Letter U7-C-STP-NRC-100036, dated February 10, 2010) about how various steel and concrete elements of site-specific structures are designed, and the design results, the applicant provided some analysis and design information. The applicant also referred to the Supplement 2 response to Question 03.07.01-13 (Letter U7-C-STP-NRC-090230, dated 12/30/09) for pertinent design summary information. In order for the staff to conclude that the design of site-specific structures meet the requirements of GDC 2 by meeting the guidance provided in SRP 3.8.4 and 3.8.5, or otherwise, the applicant is requested to provide the following additional information:

1. The applicant states in the response that a three dimensional finite element analysis (FEA) is used for structural analysis and design of the UHS/RSW Pump House. FSAR Section 3H.6.6.1 states that analysis for the seismic loads was performed using equivalent static loads and the induced forces due to X, Y, and Z seismic excitations were combined using the SRSS method of combination. However, the applicant did not describe how the equivalent static loads due to seismic excitation were determined and applied to the static FEA model from the results of soil structure interaction (SSI) analysis used for determination of seismic response. Therefore, the applicant is requested to provide details of how seismic response analysis results from dynamic SSI analysis were transferred to the static FEA model, including how the effects of accidental torsion were included in the analysis and design of UHS/RSW Pump house. Please also update FSAR with the information, as appropriate.

2. The applicant stated in its response that the modulus of subgrade reaction for static loading was calculated as the average of the local values at nine locations under the foundation. The applicant is requested to provide these nine values, and explain why it is considered appropriate to use the average value. Please also explain how the foundation subgrade modulus was used for calculating nodal springs for the FEA model, and how the effect due to coupling of soil springs was considered in the analysis.

3. For seismic loading, the applicant has outlined a hand-calculated procedure that utilizes published formulas and charts to estimate the foundation spring constants. According to this procedure, the equivalent modulus and Poisson's ratio of a layered soil system are first estimated using the cumulative strain energy method. The resulting values are then used in the equations for computation of the spring constants for a rigid foundation of an arbitrary shape embedded in a uniform half-space. The shear moduli used for individual layers are strain compatible values, and include the mean, upper bound, and lower bound soil cases. The approximate procedure outlined above for developing the foundation spring constants does not take into account the pressure distribution under the base slab. Furthermore, this procedure does not account for the frequency dependence of

these springs. As such, the applicant is requested to provide a justification for not considering the effects of pressure distribution and system frequency in developing the foundation dynamic springs including describing the impact on the calculated results.

4. The applicant's response does not provide details as to how the soil springs calculated under static and seismic loadings are inputted to the 3-D static FEA model to calculate the design stresses. Therefore, the applicant is requested to describe in detail how the static and seismic soil springs are inputted into the FEA model, and how the results are obtained for stress evaluations. Specifically, the applicant is requested to explain if the two sets of springs were used in a single model, and how the two sets were combined to a single set of springs. Otherwise, if the two sets of springs were applied to separate FEA models, describe how the load combinations were performed. The applicant is also requested to provide sufficient detail to assist staff in understanding how static and seismic soil springs are used in the FEA model and results combined for stress evaluations.

5. In the FSAR mark-up of Sections 3H.6.6.3.1 and 3H.6.6.3.2 provided with the response, the applicant identifies the method used by the applicant for combining forces and moments. In this method, for each reinforcing zone, the maximum force or moment is coupled with the corresponding moment or force for design for the same load combination. It is not clear if this method of combining forces and moments for design will envelop the worst combination of forces and moments for all elements in a reinforcing zone. Therefore, the applicant is requested to describe the method of combining forces and moments used by the applicant with a typical example of a reinforcing zone, and demonstrate that this method of combination will yield the worst combination of forces and moments that should be considered for design.

6. The staff notes that in the FSAR mark-up of Section 3H.6.6.3.1 provided with the response, the reported values of soil springs for the RSW Pump House are significantly larger than those for the UHS basin. The applicant is requested to confirm these values, and explain the reason for the large difference.

7. The response did not include any information about the maximum static and dynamic bearing pressures under the foundations of UHS/RSW Pump House. The applicant is requested to provide the maximum static and dynamic bearing pressure under the foundations of UHS/RSW Pump House, compare these values with the maximum allowable static and dynamic bearing pressures, and include this information in the FSAR.

8. In its response to Question 03.07.01-19 (letter U7-C-STP-NRC-100129, dated June 7, 2010), the applicant provided analysis and design information for the seismic category I Diesel Generator Fuel Oil Storage Vault (DGFOSV) a which was not previously included in the FSAR. The information included in the response does not describe how structural analysis and design of the structure was performed. Also, reference is made to FSAR Section 3H.6.4 for design loads. FSAR Section 3H.6.4 has been updated several times in various responses, and it is not clear where this information can be found. Therefore, the applicant is requested to provide complete structural analysis and design

information for the DGFOSV to ensure it meets acceptance criteria 1 through 7 of SRP 3.8.4 and 3.8.5. The staff needs this information to conclude that the DGFOSV is designed to withstand seismic loads and meet GDC 2. Include in the response an updated version of Appendix 3H where structural analysis and design information for all seismic category I structures can be found.

9. While reviewing this response, and other responses referenced in this response, the staff noted that the applicant has used different values of coefficient of friction for sliding stability evaluation; e.g., the value 0.3 was used for the RSW Pump House, 0.4 was used for UHS basin, 0.58 was used DGFOSV, and for the Reactor Building (RB) and the Control Building (CB), it was stated to be more than 0.47. It is not clear if these values are the required coefficient of friction, or the minimum coefficient of friction available. The applicant is requested to clearly specify the minimum coefficient of friction at various locations of the site, if they are different, and explain how these values were determined. Please also clarify this information in the FSAR.

10. The staff noted references to Diesel Generator Fuel Oil Tunnel (DGFOT) in several RAI responses. Please confirm that DGFOT is not a seismic category I structure, and if it is seismic category I, include the analysis and design information to show how the design of the DGFOT meets the acceptance criteria 1 through 7 in the SRP 3.8.4 and 3.8.5 in the FSAR.

SUPPLEMENTAL RESPONSE:

The Supplement 3 response to this RAI was submitted with Nuclear Innovation North America (NINA) letter U7-C-NINA-NRC-110081, dated June 16, 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. *Determine column accelerations for column mass and hydrodynamic mass based on column frequency and spectra at top and bottom of the columns and revise RAI 03.08.04-30, supplement 1 to report new information (Audit Action Item 3.7-15, Punch List Item 17)*

   In order to account for hydrodynamic mass effect on the accelerations for column mass and column hydrodynamic mass, the nodal zero period accelerations (ZPAs) from the soil-structure-interaction (SSI) analysis for the intermediate nodes of the columns (i.e. excluding top and bottom nodes) are multiplied by the scale factors shown in Table 03.08.04-30 S4.1. These scale factors are determined as described below.

Based on column size and boundary conditions at the top and bottom supports, the UHS basin columns are idealized considering the following four column configurations:

1. 5x5 Column_BotFix_TopFixX_TopFixY
2. 5x5 Column_BotFix_TopFixX_TopPinY
3. 5x5 Column_BotFix_TopPinX_TopFixY
4. 5x12 Column_BotFix_TopPinX_TopPinY

Figure 03.08.04-30 S4.1 shows a plan view of the UHS basin with the column configuration type labeled for each column.

The calculation of column acceleration scale factors is performed with eight SAP2000 finite element models, the eight models comprising of the four configurations, each with and without the hydrodynamic mass. A response spectrum analysis is run for each of the eight models, with 7% damping. The input response spectra used in these analyses are shown in Figures 03.08.04-30 S4.2 and 03.08.04-30 S4.3, in the global X and Y directions, respectively. The response spectra shown in Figures 03.08.04-30 S4.2 and 03.08.04-30 S4.3 are the envelop spectra of all the full basin SSI analysis cases, and envelop the response spectra of top and bottom nodes of all columns that are of the same configuration. Please note that for each direction of excitation there are only three separate input motions, since for Configurations 1 and 2 the same motion is used in the X direction, and for Configurations 1 and 3 the same motion is used in the Y direction. The hydrodynamic mass applied to the columns is determined as described in Item 1 of the RAI 03.07.02-28 response, submitted with NINA letter U7-C-NINA-NRC-110043, dated March 15, 2011.

Figures 03.08.04-30 S4.4 through 03.08.04-30 S4.7 provide the comparisons of the accelerations along the columns from response spectrum analyses with and without hydrodynamic mass. Based on the results from these response spectrum analyses for each column configuration, the acceleration scale factors are computed at each interior node using the following equation:

$$\text{Acceleration Scale Factor} = \frac{\text{Acceleration\_with\_Hydrodynamic\_Mass}}{\text{Acceleration\_with\_NoHydrodynamic\_Mass}}$$

Finally, for each column configuration and each direction (i.e. X and Y), the corresponding maximum scale factor for the interior nodes is selected as the scale factor for all interior nodes on columns of that configuration. If the scale factor is less than one, conservatively a scale factor equal to one is chosen.

b. *In the manual calculation for design of Pump House roof slab, increase the vertical seismic load for PH roof based on examination of structural mesh sensitivity results (Audit Action Item 3.7-18, Punch List Item 18)*

Table 03.07.02-25.1 provided with the RAI 03.07.02-25, Revision 1 response which was submitted with NINA letter U7-C-NINA-NRC-110043, dated March 15, 2011 provides the modification factors for the maximum accelerations based on examination of the results for the structural mesh sensitivity analysis. Referring to this table, the increase in the maximum vertical acceleration for the PH roof slab is 1.26.

The modification factors for generation of response spectra, accounting for the cumulative effect of the structural and SSI mesh refinements are provided in COLA Part 2, Tier 2, Table 3H.6-16. Referring to Table 3H.6-16, for 7% damping and frequencies in excess of 30 Hz, the modification factor for the PH roof vertical acceleration is 1.409.

In the manual calculations of the PH roof slab, consistent with the modification factors for the response spectra of the PH roof, the vertical seismic loads were increased by a factor of 1.409. This modification factor of 1.409 accounts for the modification factor of 1.26. Therefore, no further adjustment is required for the design of PH roof slab.

COLA updates reflecting the results of analysis considering scaled column accelerations, as discussed in Item (a) above, will be provided, along with the revisions for shear design, later, currently scheduled for August 17, 2011 (see Punch List Item 56).

Table 03.08.04-30 S4.1: Column Acceleration Scale Factors Due to the Effect of Hydrodynamic Mass

Column Configuration	Acceleration Scale Factors X Direction	Acceleration Scale Factors Y Direction
5x5_BotFix_TopFixX_TopFixY	1.19	1.00
5x5_BotFix_TopFixX_TopPinY	1.20	1.69
5x5_BotFix_TopPinX_TopFixY	1.00	1.00
5x12_BotFix_TopPinX_TopPinY	1.03	1.37

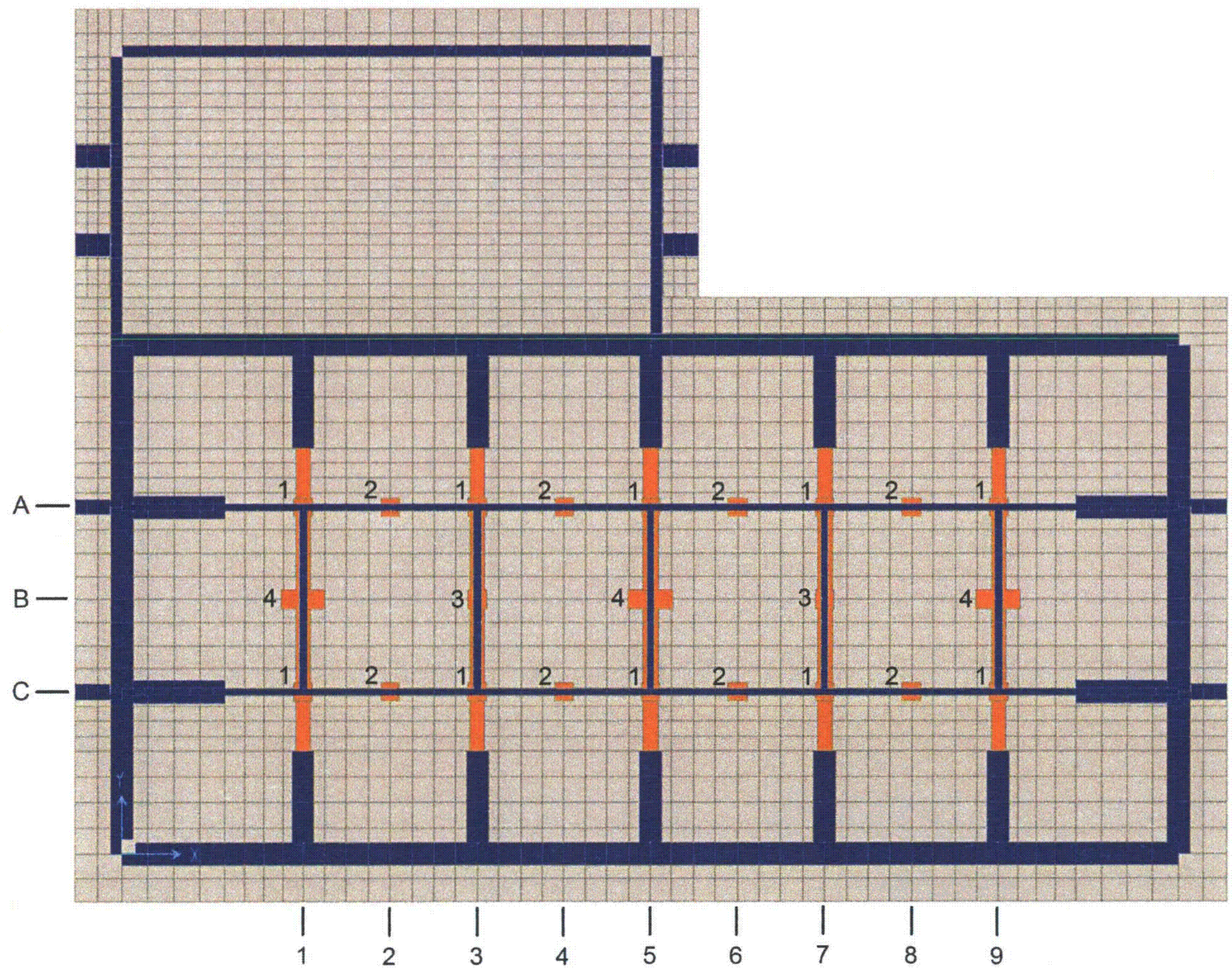

Figure 03.08.04-30 S4.1: Plan View of UHS Basin with Column Boundary Conditions Labeled

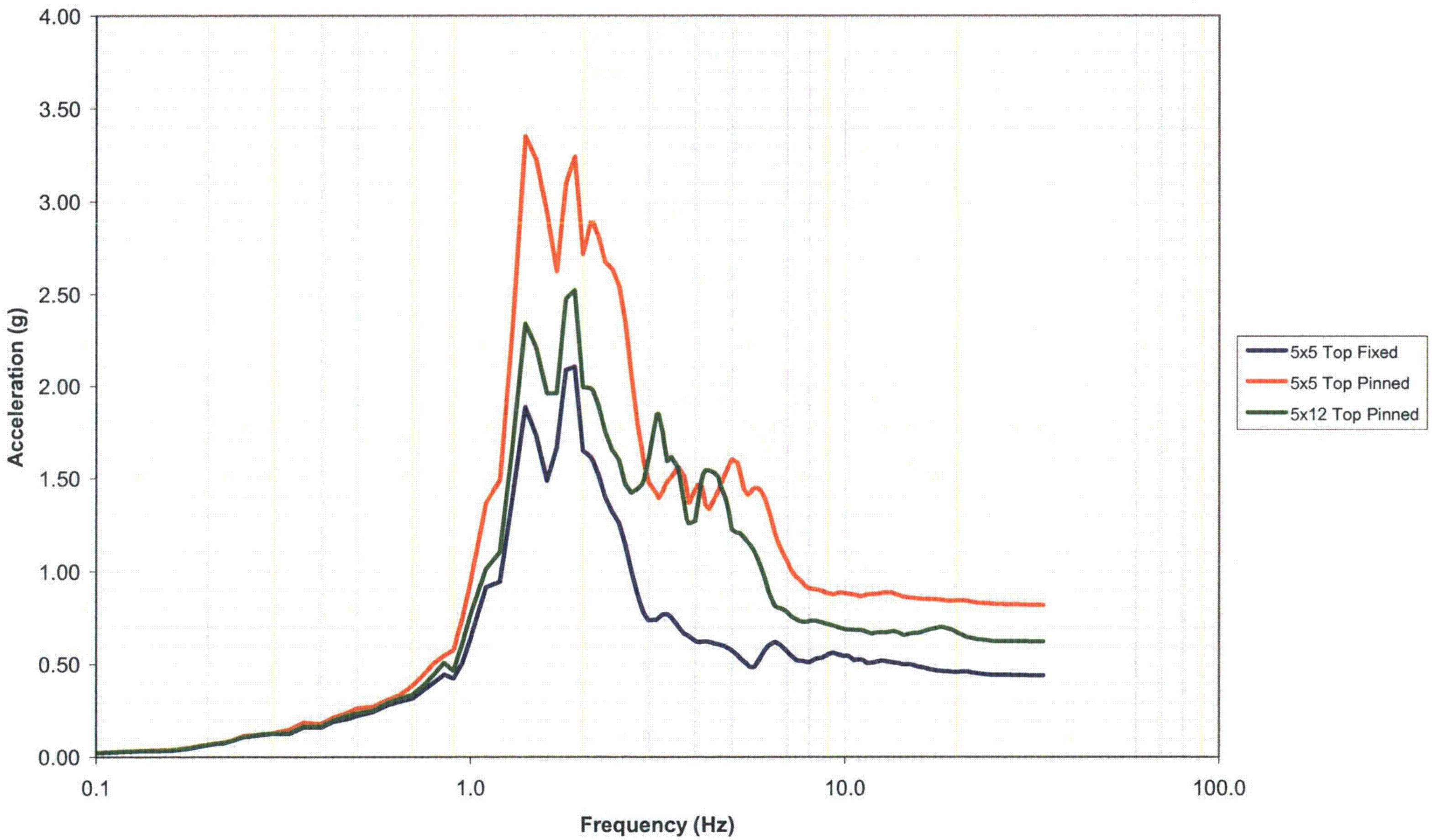

Figure 03.08.04-30 S4.2: Enveloped UHS Response Spectra for Column Top and Bottom Nodes, X-Direction, 7% Damping

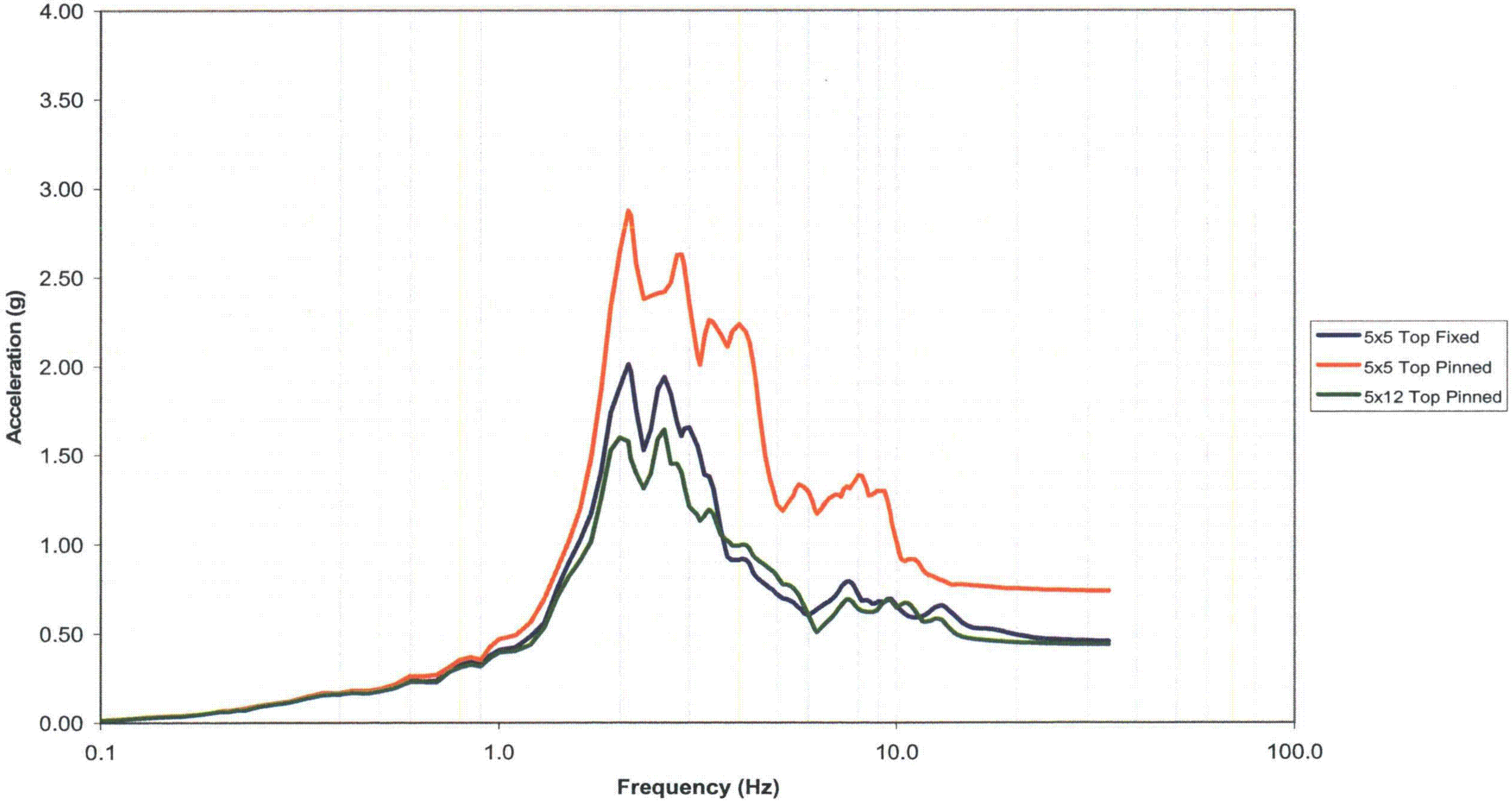

Figure 03.08.04-30 S4.3: Enveloped UHS Response Spectra for Column Top and Bottom Nodes, Y-Direction, 7% Damping

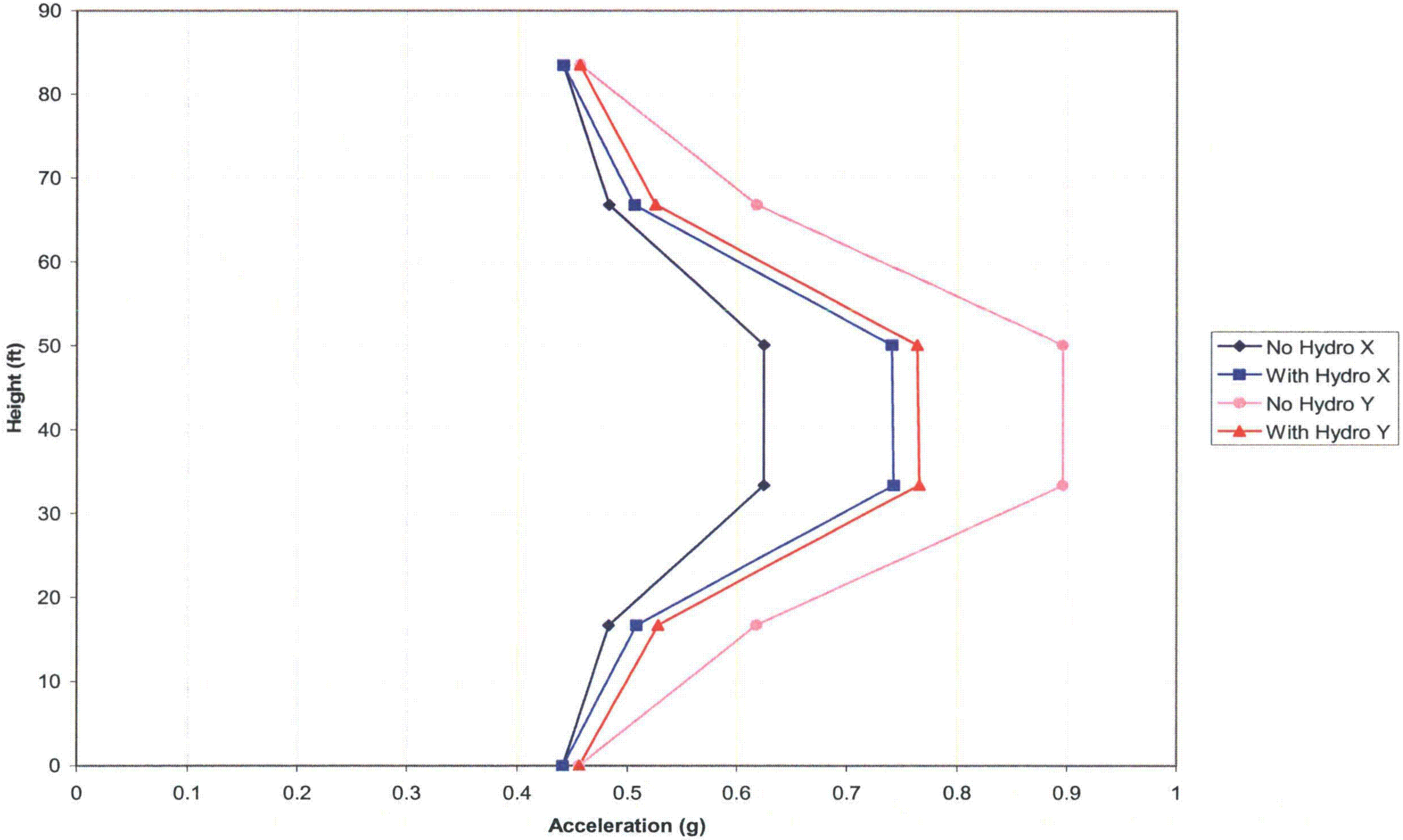

Figure 03.08.04-30 S4.4: Configuration 1 Column Acceleration Comparison, No Hydrodynamic Mass vs. With Hydrodynamic Mass

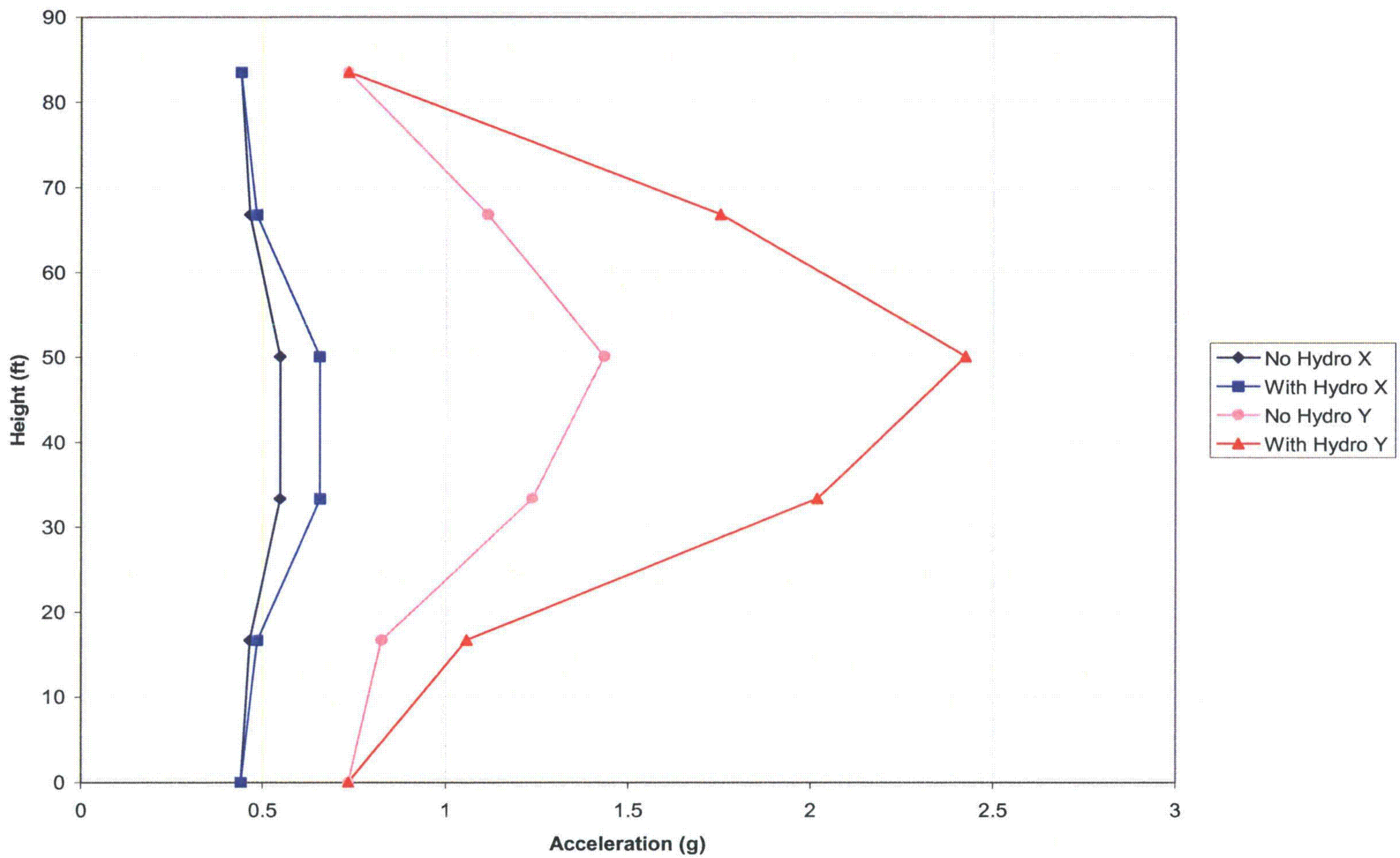

Figure 03.08.04-30 S4.5: Configuration 2 Column Acceleration Comparison, No Hydrodynamic Mass vs. With Hydrodynamic Mass

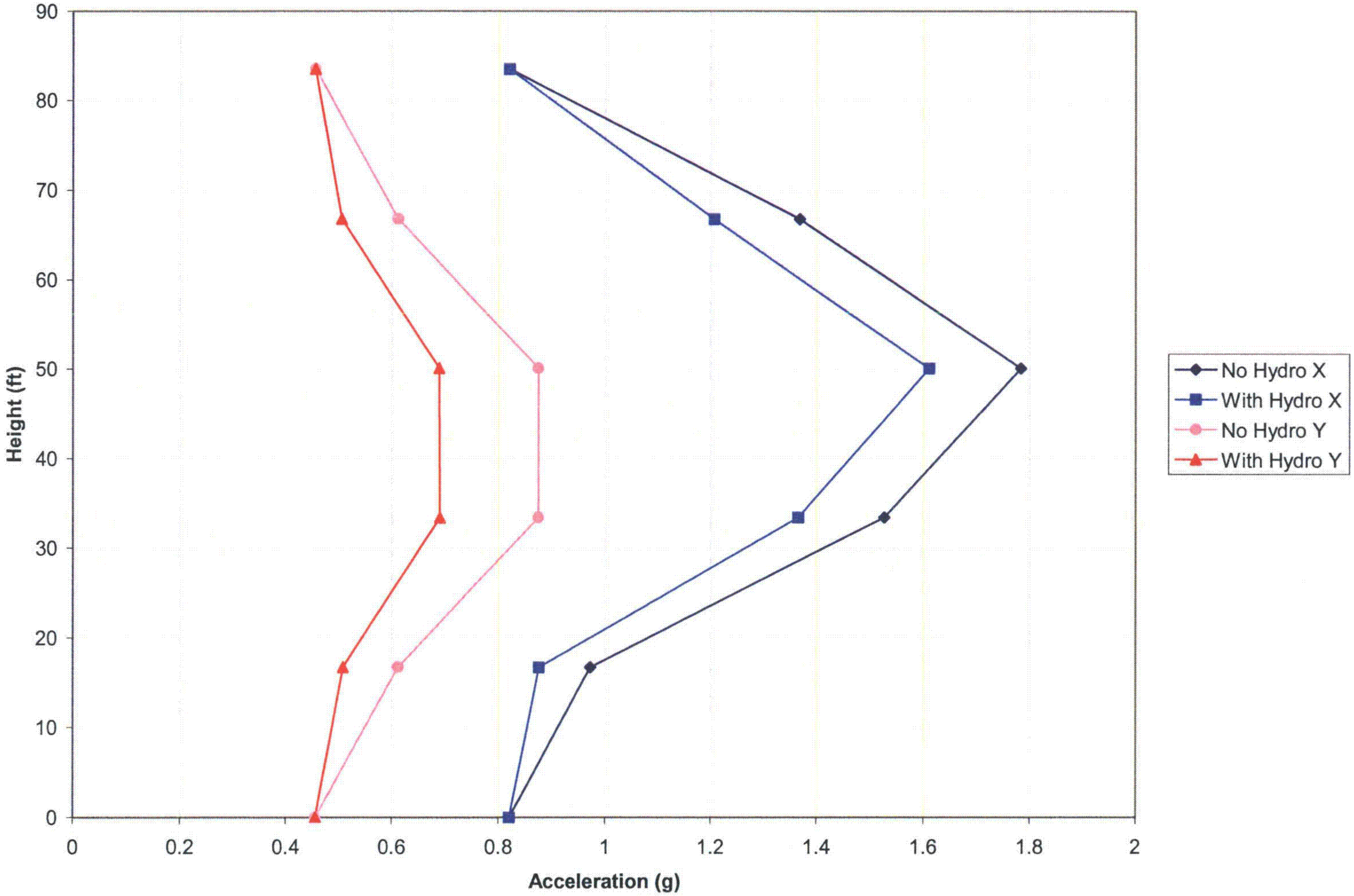

Figure 03.08.04-30 S4.6: Configuration 3 Column Acceleration Comparison, No Hydrodynamic Mass vs. With Hydrodynamic Mass

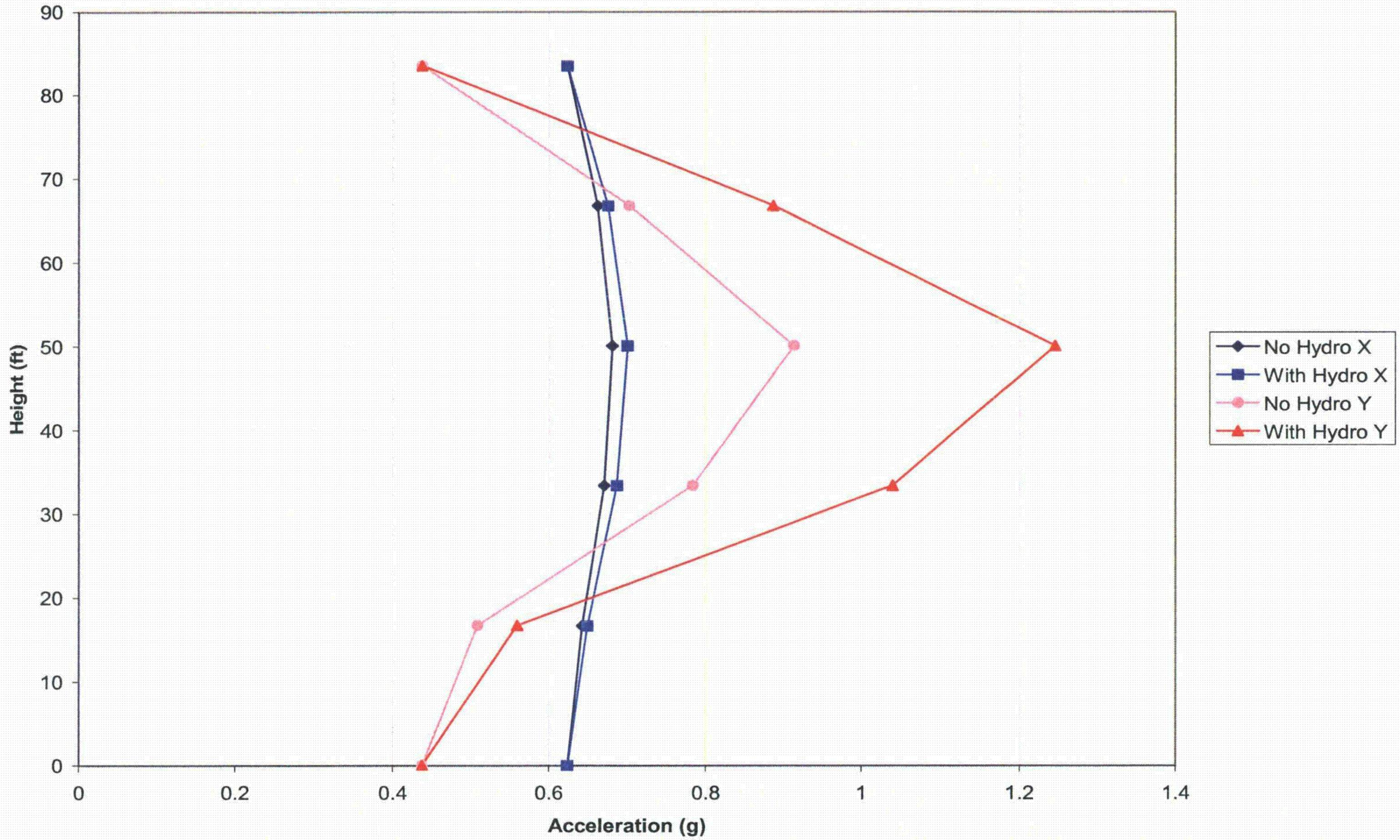

Figure 03.08.04-30 S4.7: Configuration 4 Column Acceleration Comparison, No Hydrodynamic Mass vs. With Hydrodynamic Mass

RAI 03.08.04-35, Revision 1

QUESTION:

10 CFR 50, Appendix A, GDC 2, requires that structures important to safety shall be designed to withstand the effects of natural phenomena with appropriate combination of the effects of normal and accident conditions. To meet this requirement, bearing pressure under the basemat of seismic category I structures under all design loading combinations must be within the allowable bearing capacity for a site. During the October 2010 Audit the applicant presented the procedures used to determine the dynamic soil pressures beneath the UHS/PH foundation mat, resulting from SSE loadings. In this procedure, the applicant applied vertical and lateral loads to the structure to compute equivalent eccentricity of the vertical load. The applicant then considered a reduced bearing area of the basemat accounting for the computed eccentricity of the vertical load over which the vertical load is concentric. The soil bearing pressures are calculated as uniformly distributed pressure under the reduced foundation area. The applicant then calculated a factor of safety (FOS) as the quotient between the total ultimate soil bearing capacity and the calculated bearing pressure.

The staff noted that the applicant's methodology of calculating soil bearing pressures (based on an equivalent foundation and uniformly distributed soil pressures) under the foundations was not consistent with the analysis and design of the structures including basemat (based on SAP2000 models with soil spring elements), and may significantly underestimate the expected foundation toe pressures for loading combinations having large overturning moment. Therefore, the staff requests the applicant to provide additional information describing how the procedure used by the applicant for verifying soil bearing pressures reconcile with the analysis and design (i.e. internal element forces, displacements, total building tilt, soil settlement, etc.) of the structures and foundations for all design load combinations, including those where foundation uplift may be present.

REVISED RESPONSE:

The original response to this RAI was submitted with Nuclear Innovation North America (NINA) letter U7-C-NINA-NRC-110050, dated April 5, 2011. This revision provides the response to the following action item discussed in the NRC audit performed during the week of May 23, 2011.

- Revise response to RAI 03.08.04-35 to explain design in lieu of further analysis for equivalent bearing pressure (Audit Action Item 3.8-38, Punch List Item 93)

The revised portions of the response are marked with revision bars.

The methodology used for foundation (soil) bearing capacity evaluation and determination of corresponding safety factors is in accordance with that described in COLA

Section 2.5S.4.10.3. This methodology for evaluation of eccentrically loaded foundations was developed by Prof. J. Brinch Hansen and Prof. G.G. Meyerhof, and is well-established in geotechnical manuals and textbooks.

In this methodology, the coupled moment and the vertical load acting simultaneously at the center of the foundation are transformed to an equivalent foundation loading system with the same vertical load solely acting at a point offset from the center of actual foundation. The offset distances, defined as eccentricities, are calculated as follows:

$$e_x = \frac{M_x}{F_z} \qquad \text{Equation 1}$$

$$e_y = \frac{M_y}{F_z} \qquad \text{Equation 2}$$

$$B' = B - 2e_x, \qquad \text{Equation 2.5S-24B (COLA)}$$

$$L' = L - 2e_y, \qquad \text{Equation 2.5S-24B (COLA)}$$

where;

- e_y = eccentricity of load in x-direction (parallel to L),
- e_x = eccentricity of load in y-direction (parallel to B),
- M_y = moment about the y-axis in an x-y coordinate system,
- M_x = moment about the x-axis in an x-y coordinate system,
- F_z = vertical force acting perpendicular to the x-y plane,
- B = foundation width in y-direction,
- L = foundation length in x-direction,
- B' = effective foundation width, and
- L' = effective foundation length.

The effective foundation area in terms of the reduced foundation width and length (B' and L'), and the soil properties are used in the equation for calculating the ultimate bearing capacity as a pressure, q_{ult}, in COLA Part 2, Tier2 Equation 2. 5S.4-15 and associated components in the equation given by subsequent COLA Part 2, Tier2 Equation 2.5S.4-15B through Equation 2.5S.4-21A.

The factor of safety of the foundation is expressed as a ratio of ultimate load (q_{ult} x B' x L') to the applied vertical load (Fz), in COLA Part 2, Tier2 Equation 2. 5S.4-22. The factors of safety in COLA Part 2, Tier2 Table 2. 5S.4-41C are based on applied design load combinations acting on the foundations, including those where foundation uplift may be present.

In the 1970 publication of his work Hansen (Reference 1) describes that the eccentricity is "... best taken into account by considering the so-called effective foundation area ...". The "effective foundation area" is positioned so that its geometric center coincides with the new (offset) load center.

In Reference 1, Hansen states "Meyerhof, the writer and others have shown that the actual bearing capacity of an eccentrically loaded foundation will be very nearly equal to the bearing capacity of the centrally loaded effective foundation area".

This concept of using effective foundation area with a centrally applied vertical load to represent the actual foundation area subject to the same vertical load plus moments is widely recommended in geotechnical manuals and textbooks and it implicitly accounts for the non-uniform pressure distribution under the actual foundation, including the heel and toe pressures.

The design of structures, including their basemats, is based on finite element analyses where the foundation (soil) is represented by soil springs. Since the foundation (soil) is represented by soil springs the pressure distribution at the bottom of the basemat under eccentric loading will vary, and thus the design of the structure, including its basemat, will appropriately account for higher heel and toe pressures noted by the NRC staff.

In summary, the effective foundation area method has been used to compute the factor of safety against dynamic bearing capacity, and the finite element method (with soil springs) has been used for design of structure and basemat. These methodologies are appropriate for their intended application. Both of these methodologies appropriately account for higher heel and toe pressures under eccentric loading. Therefore, the foundation bearing capacity check using effective foundation area, as described above, and design of structures including their foundations using finite element analysis, as described above, are adequate and no further analysis is required.

No COLA change is required as a result of this response.

References:

1. Hansen, J.B., 1970, A Revised and Extended Formula for Bearing Capacity. *Bulletin of the Danish Geotechnical Institute*, No. 28, pp. 5-11.