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April 5, 2011
U7-C-NINA-NRC-110050

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
<u>Response to Request for Additional Information</u>

Attached are Nuclear Innovation North America LLC (NINA) responses to NRC staff questions included in Request for Additional Information (RAI) letter number 376 related to Combined License Application (COLA) Part 2, Tier 2, Section 3.8.4. The attachments provide the responses to the RAI questions listed below:

03.08.04-34 03.08.04-35 03.08.04-36

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 4/5/11

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

jep

Attachments:
1. RAI 03.08.04-34
2. RAI 03.08.04-35
3. RAI 03.08.04-36

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NRO

STI 32845525

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

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RAI 03.08.04-34

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, all seismic category I structures must be designed for required strength at all locations in the structure. During the October 2010 Audit the applicant presented the procedures to verify the concrete sections of the UHS/PH structural members resulting from the code-required load combinations. The internal forces (i.e. shear, moment, axial force, torsion, etc.) used to determine the required strength of the structural members (i.e. walls, slabs, beam, columns, etc.) of the UHS/PH building are generated by the applicant with the help of SAP2000 models simulating the building's static and dynamic behavior. These element forces are subsequently processed by the applicant with a number of in-house developed programs for design of concrete sections. It was noted that concrete slabs and walls were designed for out-of-plane shear by averaging the element shear forces across cut lines that extended along the entire width of the walls and slabs. The staff considers that averaging of out of plane shear along the entire cut line of a slab or wall could lead to unconservative estimate of shear stress in slabs. The subject was discussed with the applicant during the audit. Although the applicant explained the procedure by referencing to ACI 349-97, Section 11.12, "Special provisions for slabs and footings," it did not provide the staff with a sufficient interpretation of the provision of the ACI code, which appears to be intended for shear strength of slabs and footings in the vicinity of columns, concentrated loads, or reactions, to close this issue. ACI 349- 97, Section 13.3.1, states that a slab system may be designed by any procedure satisfying conditions of equilibrium and geometric compatibility, if shown that the design strength at every section is at least equal to the required strength. Averaging of out-of-plane shear across the entire width of a slab may not show that the design strength at every section is at least equal to the required strength. Therefore, in order for the staff to conclude that the site-specific structures are adequately designed for out-of plane shear, the staff requests STP to demonstrate that use of average shear force across the entire width of slab, instead of the shear force demand at every section obtained from analysis may be considered acceptable by any or more of the following:

- Obtain clarification from the ACI regarding validity of use of Section 11.12 of ACI 349-87 for the situations where the provisions of the code were used,
- Provide examples of any precedence where similar methodology was accepted by the staff,
- Provide detailed justification using industry accepted standards, technical references, experimental results, etc., to justify redistribution of the shear forces obtained from finite element analysis.

The applicant is also requested to update the FSAR as necessary.

RESPONSE:

The following is based on the discussions with the NRC staff during the audit of March 14-18, 2011. Since the ACI 349-97 code does not provide clear guidelines for out-of-plane shear design when loads are obtained from a finite element analysis, the design of the site-specific Seismic Category I structures, Radwaste Building, and Diesel Generator Fuel Oil Tunnels will be revised such that the design is conservatively based on the out-of-plane shear obtained for each element from finite element analysis, without averaging the shear over several elements. The results of these revised designs are currently scheduled to be available for NRC review in August, 2011.

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

RAI 03.08.04-35

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.

RESPONSE:

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, Tier 2 Equation 2.5S.4-15 and associated components in the equation given by subsequent COLA Part 2, Tier 2 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, Tier 2 Equation 2.5S.4-22. The factors of safety in COLA Part 2, Tier 2 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.

The above methodology for determination of safety factors for foundation (soil) bearing capacity and the design of structures, including their basemat, were discussed with the NRC staff during the NRC audit of March 14-18, 2011, and it was agreed that calculation of the safety factors for the foundation (soil) bearing capacity and design of the structures based on finite element analysis are acceptable. However, the NRC staff requested an analysis demonstrating that the basemats of these structures will be adequate for the pressure distribution under the basemat with the effective foundation area for eccentrically loaded condition. Furthermore, it was agreed that for this confirmatory analysis, which does not represent a real loading condition, the basemat without any soil springs could be evaluated for the vertical loads from the structure and the soil pressure from the effective foundation area by fixing the basemat edges to prevent model instability.

In response to this staff request, a confirmatory analysis will be performed for the basemats of the Ultimate Heat Sink/Reactor Service Water Pump House (UHS/RSW Pump House) and Diesel Generator Fuel Oil Storage Vaults (DGFOSV), for the loading corresponding to the lowest safety factor for the soil bearing pressure. Since the basemats of the Diesel Generator Fuel Oil tunnels and RSW Piping tunnels are mainly one way slabs with short spans and are lightly loaded, they require no such confirmatory analysis.

The results of the above noted confirmatory analysis for basemats of the UHS/RSW Pump House and DGFOSV are currently scheduled to be provided in June, 2011.

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.

RAI 03.08.04-36

QUESTION:

Follow-up Question to Question 03.08.04-33(Question 18287)

The staff reviewed the applicant's response to question 03.08.04-33 regarding acceptability of using newer versions of ACI 349 and ASME Section III, Division 2, than those used in the ABWR DCD. It was noted that the applicant identified several areas in the newer versions of both codes where the newer codes are either more restrictive or may result in more robust design. However, the applicant did not demonstrate how the design information included in the ABWR DCD that is being incorporated by reference is affected by the provisions of the newer codes in these cases. When taking a departure, the applicant must evaluate that departure against other information in the FSAR (including the DCD incorporated by reference), to determine whether the departure is acceptable in light of the rest of the FSAR and whether the departure is consistent with the rest of the FSAR, and make any appropriate changes as a result of this evaluation. The applicant is requested to evaluate any potential adverse impact of the provisions of the newer codes that are more restrictive, or result in a more robust design, on the ABWR DCD structural design information. Alternatively, the applicant may continue using the earlier versions of the codes that were used for certification of the ABWR design.

RESPONSE:

The attached Table 03.08.04-36.1 provides an evaluation of any impact on the design information included in the DCD that is being incorporated into the COLA by reference due to the use of newer versions of ACI 349 and ASME, Section III, Division 2 codes, as proposed by Departure STD DEP 1.8-1. The proposed code years being evaluated are the 1997 version of ACI 349 and ASME, Section III, Division 2 Edition 2001 with 2003 addenda. The effect of any changes in code requirements is addressed in the attached table. Based on this evaluation it is concluded that there is no impact on the DCD design information due to the proposed Code year changes.

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

Table 03.08.04-36.1: Evaluation of DCD Sections for Impact Due to ACI 349 and ASME, Section III, Div. 2 Code Year Change		
DCD Section/Table/Figure	Contents	Justification for No Impact due to Code Year Change
Section 3.8.1.5	Indicates the maximum and allowable tangential shear stresses, as well as the actual shear strain in the RCCV.	The allowable stress is a DCD requirement per Table 3.8-2 and is not a code requirement. The calculated maximum stresses and strains are not affected by the changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years.
Section 3H.1.5.5.1.1	Indicates the maximum rebar stresses as well as the actual liner strain in the RCCV's containment wall. The data is based on Tables 3H.1-18 and 3H.1-19.	This section just reports data contained in other DCD Tables, and does not report allowables. See discussions for referenced Tables 3H.1-18 and 3H.1-19. The calculated maximum stresses and strains are not affected by the changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years.
Section 3H.1.5.5.1.2	Indicates the maximum rebar stresses as well as the actual liner strain in the RCCV's top slab. The data is based on Tables 3H.1-18 and 3H.1-19.	This section just reports data contained in other DCD Tables, and does not report allowables. See discussions for referenced Tables 3H.1-18 and 3H.1-19. The calculated maximum stresses and strains are not affected by the changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years.
Section 3H.1.5.5.1.3	Indicates the maximum rebar stresses as well as the actual liner strain in the RCCV's mat. The data is based on Tables 3H.1-18 and 3H.1-19.	This section just reports data contained in other DCD Tables, and does not report allowables. See discussions for referenced Tables 3H.1-18 and 3H.1-19. The calculated maximum stresses and strains are not affected by the changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years.
Section 3H.1.5.5.2.1	Indicates the maximum rebar stresses as well as the actual liner strain in the RCCV's diaphragm floor. The data is based on Tables 3H.1-18 and 3H.1-19.	This section just reports data contained in other DCD Tables, and does not report allowables. See discussions for referenced Tables 3H.1-18 and 3H.1-19. The calculated maximum stresses and strains are not affected by the changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years.
Section 3H.1.5.5.3.1	Indicates the maximum rebar stresses in the Reactor Building's exterior walls. The data is based on Table 3H.1-18.	This section just reports data contained in other DCD Tables, and does not report allowables. The calculated maximum stresses are not affected by the changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years.

Table 03.08.04-36.1 (continued): Evaluation of DCD Sections for Impact Due to ACI 349 and ASME, Section III, Div. 2 Code Year Change

DCD Section/Table/Figure	Contents	Justification for No Impact due to Code Year Change
Section 3H.1.5.5.3.2	Indicates the maximum rebar stresses in the Reactor Building's fuel pool girders. The data is based on Tables 3H.1-17 and 3H.1-18.	This section just reports data contained in other DCD Tables, and does not report allowables. The calculated maximum stresses are not affected by the changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years.
Table 3H.1-14	Includes percentage of reinforcing used in the analysis for representative sections of the Reactor Building and RCCV.	The percentage of reinforcing considered for the analysis is not affected by changes in code years. The required amount of reinforcing is dependent on the load combinations and load and phi factors for ACI 349 and on the load combinations and allowable stresses for ASME Div. 2. These items have not changed for major structural elements because of the code year revisions.
Table 3H.1-15	Calculated and allowable reinforcing steel and concrete stresses for RCCV design sections (including basemat immediately outside the RCCV shell) for Load Combination 1 (Pressure Test - Service Allowables)	The calculated reinforcing steel and concrete stresses are not affected by changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years. The allowable reinforcing steel stress reported is based on ASME Section III, Division 2, CC-3432.1 and the allowable concrete stress reported is based on Table CC-3431-1. Neither is affected by the code year revision.
Table 3H.1-16	Calculated and allowable reinforcing steel and concrete stresses for RCCV (and Reactor Building) design sections for Load Combination 8 (Abnormal - Factored Allowables)	The calculated reinforcing steel and concrete stresses are not affected by changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years. The allowable reinforcing steel stress reported is based on ASME Section III, Division 2, CC-3422.1 and the allowable concrete stress reported is based on Table CC-3421-1. Neither is affected by the code year revision. The allowable reinforcing and concrete stresses is dependent on the load combinations and load and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.

Table 03.08.04-36.1 (continued): Evaluation of DCD Sections for Impact Due to ACI 349 and ASME, Section III, Div. 2 Code Year Change

DCD Section/Table/Figure	Contents	Justification for No Impact due to Code Year Change
Table 3H.1-17	Calculated and allowable reinforcing steel and concrete stresses for RCCV (and Reactor Building) design sections for Load Combination 15 (Abnormal LBL plus SSE - Factored Allowables)	The calculated reinforcing steel and concrete stresses are not affected by changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years. The allowable reinforcing steel stress reported is based on ASME Section III, Division 2, CC-3422.1 and the allowable concrete stress reported is based on Table CC-3421-1. Neither is affected by the code year revision. The allowable reinforcing and concrete stresses is dependent on the load combinations and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.
Table 3H.1-18	Calculated and allowable reinforcing steel and concrete stresses for RCCV (and Reactor Building) design sections for Load Combination 15a and 15b (Abnormal IBL and SBL plus SSE - Factored Allowables)	The calculated reinforcing steel and concrete stresses are not affected by changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years. The allowable reinforcing steel stress reported is based on ASME Section III, Division 2, CC-3422.1 and the allowable concrete stress reported is based on Table CC-3421-1. Neither is affected by the code year revision. The allowable reinforcing and concrete stresses is dependent on the load combinations and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.
Table 3H.1-19	Calculated maximum and allowable liner strains for RCCV design sections for Load Combinations 1, 8, and 15a-15b.	The calculated maximum liner strains are not affected by changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years. The allowable liner strains are based on ASME Section III, Division 2, Table CC-3720-1 and are not affected by changes in code years either.
Figure 3H.1-29	Plans and sections including the basic reinforcements in the fuel pool girders and slabs, without indication of development or lap lengths	The calculated reinforcing requirements are not affected by changes in code years because there has been no change in the loads, load combinations, or evaluation methodology between code years. The allowable reinforcing and concrete stresses are dependent on the load combinations and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.

Table 03.08.04-36.1 (continued): Evaluation of DCD Sections for Impact Due to ACI 349 and ASME, Section III, Div. 2 Code Year Change

DCD Section/Table/Figure	Contents	Justification for No Impact due to Code Year Change
Figure 3H.1-30	Rebar arrangement of RCCV wall sections	Design of the RCCV wall sections is governed by the factored load combinations (as can be judged from Tables 3H.1-15 through 3H.1-18). The concrete and reinforcing steel design requirements and stress allowables for both main reinforcing bars and shear ties for factored loads are not affected by the changes in code years.
Figures 3H.1-31 & 3H.1-32	Rebar arrangement around RCCV wall openings	Since the design of the general RCCV wall sections is governed by the factored load combinations (as can be judged from Tables 3H.1-15 through 3H.1-18), the design stresses of the RCCV wall section around the openings, with stress concentrations, will also be governed by the factored load combinations. The concrete and reinforcing steel design requirements and stress allowables for both main reinforcing bars and shear ties for factored loads are not affected by the changes in code years.
Figure 3H.1-33	Rebar arrangement of RCCV top slab	Design of the RCCV top slab sections is governed by the factored load combinations (as can be judged from Tables 3H.1-15 through 3H.1-18). The concrete and reinforcing steel design requirements and stress allowables for both main reinforcing bars and shear ties for factored loads are not affected by the changes in code years.
Figures 3H.1-34 & 3H.1-35	Rebar arrangement of Reactor Building (including RCCV) basemat, Sheets 1 & 2.	The main reinforcing steel required for very thick foundation mats can be close to the minimum requirement. The minimum reinforcing steel ratio required for control of concrete cracking from the effects of shrinkage, temperature, and membrane tension given in ASME Section III, Division 2, CC-3535 was rolled back from 0.0021 in the 1989 edition to 0.0020 in the 2001 edition. Per Table 3H.1-14, the minimum rebar ratio used in the analysis for the basemat sections was 0.276% (=0.00276) which is substantially greater than both code version minimums and thus will not be affected by the code requirement change. Further more, judging from Tables 3H.1-15 through 3H.1-18, the main rebar and shear tie designs for the basemat are governed by the factored load combinations which are not affected by the changes in code years. For the Reactor Building (outside RCCV), the minimum reinforcing for the basemat per ACI 349 is 0.0018 for both code years, though the trigger point for this requirement varies. The ratio provided per this figure is 0.0020, so the code year change has no effect.

Table 03.08.04-36.1 (continued): Evaluation of DCD Sections for Impact Due to ACI 349 and ASME, Section III, Div. 2 Code Year Change

DCD Section/Table/Figure	Contents	Justification for No Impact due to Code Year Change
Figure 3H.1-36	Rebar arrangement of diaphragm floor.	Design of the diaphragm floor slab is governed by the factored load combinations (as can be judged from Tables 3H.1-15 through 3H.1-18). The concrete and reinforcing steel design requirements and stress allowables for both main reinforcing bars and shear ties for factored loads are not affected by the changes in code years.
Figure 3H.1-37	Rebar arrangement of Reactor Building shear walls.	The calculated reinforcing requirements are not affected by changes in code years. The allowable reinforcing and concrete stresses are dependent on the load combinations and load and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions. In addition, the reinforcing shown in this figure meets the minimum requirements of both ACI 349-80 and ACI 349-97.
Table 3H.2-3	Required and actual mat, floor, and slab top/bottom reinforcing and shear ties	The calculated reinforcing requirements are not affected by changes in code years. The allowable reinforcing and concrete stresses are dependent on the load combinations and load and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.
Table 3H.2-4	Required and actual wall vertical and horizontal reinforcing and shear ties	The calculated reinforcing requirements are not affected by changes in code years. The allowable reinforcing and concrete stresses are dependent on the load combinations and load and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.
Figures 3H.2-21, 3H.2-22, 3H.2-23, 3H.2-24, 3H.2-25, 3H.2-26, and 3H.2-27	Plan drawings including the basic reinforcements in the mat and slabs, without indication of development or lap lengths	The calculated reinforcing requirements are not affected by changes in code years. The allowable reinforcing and concrete stresses are dependent on the load combinations and load and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.
Detail 1 of Fig.3H.2-29, Sections 1 and 2 of Figure 3H.2-30	Details to show the basic reinforcing arrangements in the mat, slabs and walls without indication of development or lap lengths	The calculated reinforcing requirements are not affected by changes in code years. The allowable reinforcing and concrete stresses are dependent on the load combinations and load and phi factors for ACI 349. These items have not changed for major structural elements because of the code year revisions.