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July 12, 2011
U7-C-NINA-NRC-110099

U. S. Nuclear Regulatory Commission
Attention: Document Control Desk
One White Flint North
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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). Attached is a supplemental response to NRC staff question included in Request for Additional Information (RAI) 03.08.04-30 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 7/12/11

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

jep

Attachment:
RAI 03.08.04-30, Supplement 5

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NRO

STI 32899907

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

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RAI 03.08.04-30, Supplement 5

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 4 response to this RAI was submitted with Nuclear Innovation North America (NINA) letter U7-C-NINA-NRC-110087, dated June 28, 2011. This supplement provides the response to the following action item discussed in the NRC audit performed during the week of May 23, 2011.

Revise the wave propagation calculations for the tunnels using the following:

- *An apparent wave velocity of 3000 ft/sec*
- *Maximum ground velocity based on site-specific SSE maximum ground acceleration of 0.13g*
- *A triangular pressure distribution limited by the maximum passive pressure*

Also revise COLA to reflect these changes (Audit Action Item 3.8-34, Punch List Item 82)

Calculation of the axial tensile strains, forces and moments at the tunnel bends, and soil pressures on the transverse leg of the tunnels near the bends due to seismic wave propagation for the Reactor Service Water (RSW) Piping Tunnels and Diesel Generator Fuel Oil Tunnels (DGFOT) have been revised using the following parameters:

- An apparent wave velocity of 3000 ft/s based on Section C3.5.2.1 of ASCE 4-98.
- A maximum ground velocity of 6.24 in/sec (which is based on 48 in/sec/g and site-specific SSE maximum ground acceleration of 0.13g).
- A triangular soil pressure distribution on the transverse leg of the tunnel near the bend which is limited by the maximum passive pressure using passive pressure coefficient Kp = 3.

COLA changes due to this response are provided in the Enclosure.

Enclosure

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.6.6.2.2 RSW Piping Tunnels

The maximum curvature is computed based on Equation 3.5-3 in Section 3.5.2.1.3 of ASCE 4 98. The maximum curvature is then converted into additional axial strain by multiplying the curvature by the distance from the centroid of the RSW Piping Tunnels to the extreme fiber of the RSW Tunnel. For these computations, the following parameters are considered:

- ~~An Rayleigh waves with~~ apparent wave velocity of 3,000 ft/sec (as recommended in Section C3.5.2.1 of ASCE 4-98)
- A maximum ground velocity of 6.24 in/sec (which is based on 48 in/sec/g and site-specific SSE maximum ground acceleration of 0.13g)~~ Conservative ground acceleration of 0.21g~~
- A triangular soil pressure distribution on the transverse leg of the tunnel near the bend which is limited by the maximum passive pressure using passive pressure coefficient Kp = 3
- ~~Maximum ground velocity of 10.08 in/sec (which is based on 48 in/sec per 1.0g ground acceleration)~~

The tensile axial strain and strain due to maximum curvature are conservatively added together to obtain the actual strain in the longitudinal direction of the RSW Tunnel. The actual strain is then compared to the cracking strain of concrete and maximum allowable strain of the reinforcing. The maximum computed tensile axial strain is 1.8~~2.9~~ x 10^{-4} in/in which is about 9~~14~~% of the rebar yield strain of 2.069 x 10^{-3} in/in. The design also accounts for the induced forces at tunnel bends due to SSE wave propagation. These forces are determined in accordance with Section 3.5.2.2 of ASCE 4-98 by considering the structure as a beam on elastic foundation. To determine the required reinforcement, the induced forces at the tunnel bends are considered to act simultaneously with all other applicable loads (including dynamic soil pressures) in the seismic load combinations.

3H.7.5.2.4 SSE Wave Propagation Effects

The design of the DGFOT accounts for the axial tensile strain and induced forces at tunnel bends due to SSE wave propagation. The axial strain on the DGFOT due to SSE wave propagation is determined based on the equations and commentary outlined in Section 3.5.2.1 of ASCE 4-98. The maximum curvature is computed based on Equation 3.5-3 in Section 3.5.2.1.3 of ASCE 4-98. ~~The induced forces at the tunnel bends are determined in accordance with Section 3.5.2.2 of ASCE 4-98 by considering the structure as a beam on elastic foundation. To determine the required reinforcement, the induced forces at the tunnel bends are considered to act simultaneously with all other applicable loads (including dynamic soil pressures) in the seismic load combinations.~~

For SSE wave propagation computations, the following parameters are considered:

- An apparent wave velocity of 3,000 ft/sec (as recommended in Section C3.5.2.1 of ASCE 4-98)
- A maximum ground velocity of 6.24 in/sec (which is based on 48 in/sec/g and site-specific SSE maximum ground acceleration of 0.13g)
- Soil pressure distribution on the transverse leg of the tunnel near the bend is limited by the maximum passive pressure using passive pressure coefficient Kp = 3

The tensile axial strain and strain due to maximum curvature are conservatively added together to obtain the actual strain in the longitudinal direction of the DGFOT. The actual strain is then compared to the cracking strain of concrete and maximum allowable strain of the reinforcing. The maximum computed tensile axial strain is 1.75×10^{-4} in/in which is about 8.5% of the rebar yield strain of 2.069×10^{-3} in/in. The design also accounts for the induced forces at tunnel bends due to SSE wave propagation. These forces are determined in accordance with Section 3.5.2.2 of ASCE 4-98 by considering the structure as a beam on elastic foundation. To determine the required reinforcement, the induced forces at the tunnel bends are considered to act simultaneously with all other applicable loads (including dynamic soil pressures) in the seismic load combinations.

Table 3H.6-6: Results of RSW Piping Tunnel Design

Location	Item	Thickness (ft)	Governing Load Combination	Design Moment (kip-ft/ft)	Design Shear (kip/ft)	Area of Reinforcement (in^2/ft)			
						Moment Reinforcement [1]		Shear Reinforcement	
						Required	Provided (both faces)	Required	Provided
Main Tunnel	Exterior Wall	3'-0"	D+Lo+F+H'+E'	226.78	36.52	1.56 (vertical)	1.56 (vertical)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	55.90	11.29	0.7 (east-west)	0.79 (east-west)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' [2]	95.22	13.16	1.13 (east-west)	1.27 (east-west)	None	None
	Basemat	3'-0"	D+Lo+F+H'+E' [2]	123.94	19.10	0.97 (east-west)	1.00 (east-west)	None	None
North End of Main Tunnel (near Control Building)	Exterior Wall	3'-0"	D+Lo+F+H'+E'	543.34	59.39	4.27 (east-west)	4.68 (east-west)	0.19	0.20
	Interior Wall	2'-0"	D+Lo+F+H'+E' [2]	152.15	19.96	1.69 (east-west)	2.25 (east-west)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	86.64	15.29	0.70 (east-west)	0.79 (east-west)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' [2]	136.30	18.03	1.49 (east-west)	2.25 (east-west)	None	None
	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	70.42	28.27	0.36 (north-south)	0.79 (north-south)	None	None
			1.4D+1.7L+1.4F+1.7H	155.74	36.39	1.16 (east-west)	1.27 (east-west)	None	None
Main Tunnel (near Access Region 1)	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	46.60	20.54	0.70 (north-south)	0.79 (north-south)	None	None

Table 3H.6-6: Results of RSW Piping Tunnel Design (Continued)

Main Tunnel (near Access Region 2)	Exterior Wall	3'-0"	D+Lo+F+H'+E'	321.96	29.22	2.21 (vertical)	2.25 (vertical)	None	None
				214.84	29.22	1.40 (horizontal)	1.56 (horizontal)	None	None
	Basemat	6'-0"	D+Lo+F+H'+E' (2)	530.76	66.74	1.66 (east-west)	2.25 (east-west)	None	None
			1.4D+1.7L+1.4F+1.7H / D+Lo+F+H'+E' (2)	500.50	66.74	1.78 (north-south)	2.25 (north-south)	None	None
Main Tunnel (near Access Region 3) North of Pump House	Exterior Wall	3'-0"	D+Lo+F+H'+E'	245.29	36.52	1.76 (vertical)	3.12 (vertical)	None	None
	Roof Slab	3'-0"	1.4D+1.7L+1.4F+1.7H	344.53	37.20	2.56 (north-south)	4.68 (north-south)	None	None
	Interior Slab	2'-0"	D+Lo+F+H'+E' (2)	150.97	19.29	1.70 (north-south)	3.12 (north-south)	None	None
	Basemat	3'-0"	1.4D+1.7L+1.4F+1.7H	236.52	38.12	1.74 (north-south)	3.12 (north-south)	0.18	0.20

Notes:

1) Unless noted otherwise, the required reinforcement in the direction not reported in the table is controlled by the minimum required reinforcement. The minimum required reinforcement for 2'-0" thick and 3'-0" thick elements is 0.36 in2/ft and 0.54 in2/ft. For such cases the provided reinforcement is 0.79 in2/ft.

2) The loading also includes loads due to internal flooding.

3) The following additional reinforcement is required due to SSE Wave Propagation:
- For the Main Tunnel, #8 bars at 12" o.c. in the longitudinal direction of the Main Tunnel for 84'-0" (measured north from the centerline of the Main Tunnel and Access Region 3)
- For Access Region 3 from 0'-0" to 56'-0" (measured east from the centerline of the intersection of the Main Tunnel and Access Region 3)
  - i. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the roof
  - ii. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the interior slab
  - iii. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the basemat
- For Access Region 3 from 56'-0" to 103'-0" (measured east from the centerline of the intersection of the Main Tunnel and Access Region 3)
  - i. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the roof
  - ii. Second layer of #11 bars at 12" o.c. in the transverse direction applied to both faces of the basemat

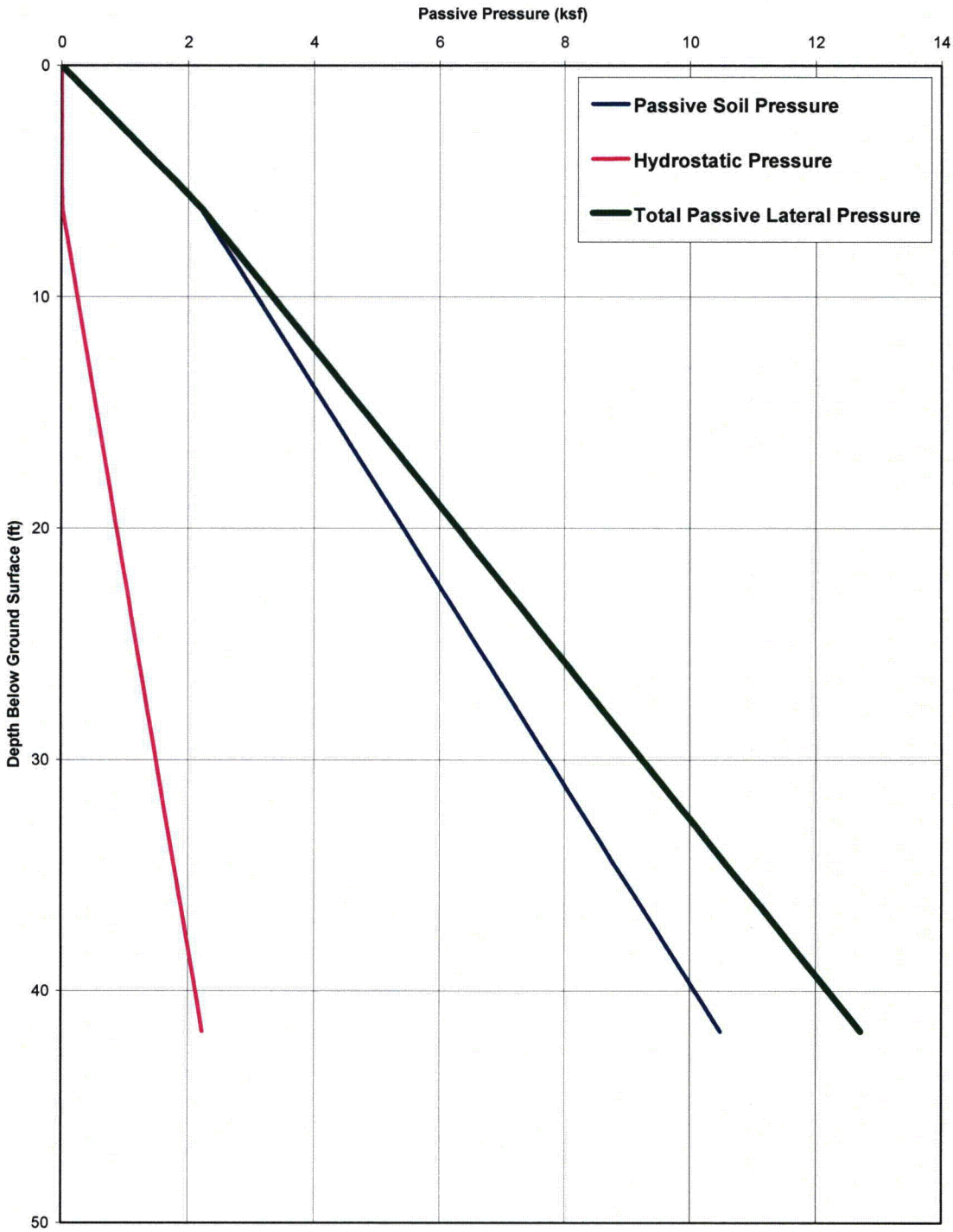

Figure 3H.6-247: Passive Lateral Earth Pressure Diagrams for Typical Section of RSW Tunnel