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G3NO-2008-00021

November 18, 2008

U. S. Nuclear Regulatory Commission
Washington, DC 20555-0001
Attention: Document Control Desk

DOCKET: No. 52-024

SUBJECT: Responses to NRC Requests for Additional Information, Letter No. 17 (GG3 COLA)

REFERENCE: NRC Letter to Entergy Nuclear, *Request for Additional Information Letter No. 17 Related to the SRP Section 3.7.1 for the Grand Gulf Combined License Application*, dated October 27, 2008 (ADAMS Accession No. ML083010058).

Dear Sir or Madam:

In the referenced letter, the NRC requested additional information on two items to support the review of certain portions of the Grand Gulf Unit 3 Combined License Application (GG3 COLA). The responses to the following Requests for Additional Information (RAIs) are provided in Attachments 1 and 2 to this letter as follows:

1. RAI Question 03.07.01-1, Ground Motion Response Spectra/Foundation Input Response Spectra (GMRS/FIRS) low frequency exceedance information

2. RAI Question 03.07.01-2, Use of Random Vibration Theory (RVT) approach

Should you have any questions, please contact me or Mr. Tom Williamson of my staff. Mr. Williamson may be reached as follows:

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This letter contains one commitment as identified in Attachment 3.

D088
LRO

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

Executed on November 18, 2008.

Sincerely,

WKH/ghd

Attachment(s):
1. Response to RAI Question No. 03.07.01-1
2. Response to RAI Question No. 03.07.01-2
3. Regulatory Commitments

cc (email unless otherwise specified):

NRC

NRC Project Manager – Grand Gulf Unit 3 COLA
NRC Project Manager – North Anna Unit 3 COLA
NRC Director – Division of Construction Projects (Region II)
NRC Regional Administrator - Region IV
NRC Resident Inspectors' Office - GGNS

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ATTACHMENT 1

G3NO-2008-00021

RESPONSE TO NRC RAI LETTER NO. 17

RAI QUESTION NO. 03.07.01-1

RAI QUESTION NO. 03.07.01-1

NRC RAI 03.07.01-1

QUESTION for Structural Engineering Branch 2 (ESBWR/ABWR Projects) (SEB2)

With respect to GGNS Unit 3 COL Application Section 3.7.1.1, Site-Specific Design Ground Motion Response Spectra, the GGNS applicant states, in part, that:

"...The site-specific GMRS/FIRS are compared with Certified Seismic Design Response Spectra (CSDRS) in Table 2.0-201. The GMRS/FIRS are enveloped by the CSDRS except for exceedance below 0.2 Hz for the horizontal motion and below about 0.15 Hz for the vertical motion. This exceedance does not have an adverse impact on the seismic design of the ESBWR Standard Plant because:

a. There are no structural frequencies below 0.2 Hz. in the frequency range of importance to structural response (frequencies greater than 0.2 Hz); the CSDRS are higher.

b. Although pools in Reactor Building/Fuel Building (RBFB) have sloshing frequencies less than 0.2 Hz, sloshing response is only a small portion of overall seismic-induced hydrodynamic loads on the pool structure and does not govern. The majority of hydrodynamic loads are due to the impulsive response of the water. Impulsive response is a function of the pool structure response at structural frequencies. The FIRS are enveloped by the CSDRS in the frequency range of importance to structural response (frequencies greater than 0.2 Hz). The impulsive response inherent in the CSDRS-based design is typically an order of magnitude higher than the sloshing response at lower accelerations of the FIRS.

c. The CSDRS for the Fire Water Service Complex (FWSC) is 1.35 times the RBFB/Control Building (CB) CSDRS. The FWSC sloshing frequency is 0.24 Hz and is enveloped by the CSDRS.

d. The higher FIRS below 0.2 Hz are irrelevant to the CB because the CB does not contain water pools.

e. The vertical excedance at frequencies below 0.15 Hz is inconsequential because vertical earthquake components do not induce sloshing.

Therefore, the adequacy of CSDRS is confirmed for Unit 3 application."

Provide response to the following items:

1. With reference to item (a) above, GGNS asserts that there are no structural frequencies below 0.2 Hz. in the frequency range of importance to structural response. Pursuant to SRP Section 3.7.1 subsection II.1.A.i and II.1.A.ii acceptance criteria, GGNS should provide pertinent quantitative analysis data to support its assertion. One example of the analysis data needed to support the GGNS assertion may include the Eigen vectors (mode shapes), Eigen values, and their corresponding modal participation factors that are

derived from the seismic analysis of GGNS RB/FB and CB structures. Provide the above data or their equivalent to validate the item (a) assertion.

2. With reference to item (b) above, discuss the methodology used in computing the sloshing frequencies and sloshing response of the pools in Reactor Building/Fuel Building (RB/FB), and list the analysis results for the pools in the RB/FB. Based on the analysis results, demonstrate that the sloshing response is only a small portion of overall seismic-induced hydrodynamic loads on the pool structure and does not govern. Explain the rationale for the GGNS statement that majority of hydrodynamic loads are due to the impulsive response of the water and compare the method used in determining the impulsive response to those used in ACI 350.3 or equivalent codes and standards. Lastly, use the RB/FB analysis results to justify the GGNS statement that the impulsive response inherent in the CSDRS-based design is typically an order of magnitude higher than the sloshing response at lower accelerations of the FIRS.

   Note: The following item is not required to be answered in your RAI response but should be resolved in the DC before the technical reviewer can conclude his safety evaluation on this section. *With reference to item (c) above, provide the technical basis for asserting that the CSDRS for the Fire Water Service Complex (FWSC) is 1.35 times the RBFB/Control Building (CB) CSDRS (The use of the 1.35 factor in defining the CSDRS for the FWSC is part of the unresolved RAI 3.7-63 of the ESBWR DC application).*

3. With reference to item (c) above, discuss the computational basis for asserting that the FWSC sloshing frequency is 0.24 Hz, thus, is enveloped by the CSDRS.

4. With reference to item (e) above, GGNS stated that the vertical excedance at frequencies below 0.15 Hz is inconsequential because vertical earthquake components do not induce sloshing. It should be noted that under the influence of vertical excitation, liquid exerts a symmetric hydrodynamic pressure on tank wall. Knowledge of this pressure is essential in properly assessing the safety and strength of tank wall against buckling. For any of the circular tanks in GGNS RB/FB, as applicable, discuss how the effect of the vertical spectral excedance at frequencies below 0.15 Hz on tank wall buckling was evaluated to support the GGNS statement that the vertical excedance at frequencies below 0.15 Hz is inconsequential.

Entergy Response

The individual questions in the RAI above have been numbered as Items No. 1 through No. 4 for ease of reference and clarity.

RAI Item No. 1:

The eigenvalue analysis performed for the DCD is applicable to Grand Gulf Nuclear Station, Unit 3 (GGNS-3). The DCD analysis is presented in DCD Tier 2, Appendix 3A, for a wide range of site conditions, from soft soil to hard rock. The soft soil results apply to GGNS-3, since the GGNS-3 shear wave velocities shown in FSAR Table 2.5.2-207 are closer to the DCD 1000 ft/sec shear wave velocity condition for a soft site. The DCD soft site eigenvalue analysis results, calculated using DAC3N computer code (DCD Tier 2 Section 3C.7.1), for modal frequencies, periods, and modal participation factors are presented in DCD Tier 2, Tables 3A.7-1 and 3A.7-8 for the RB/FB and CB, respectively. As shown, the lowest modal frequencies for structures are 1.19 Hz for RB/FB and 2.84 Hz for CB. The soft soil structural frequencies are the lowest among the soft, medium, and hard soil sites and are conservative with respect to the low frequency exceedance.

RAI Item No. 2:

The sloshing frequencies, sloshing pressures, and impulsive pressures of the pools in the RB/FB are computed in accordance with TID-4500 (22nd edition), Chapter 6 and Appendix F, "Nuclear Reactors and Earthquakes," U.S. Atomic Energy Commission, August 1963 (also known as the Housner method). The TID's rigid wall assumption for the horizontal impulsive mode is not used. In the ESBWR design, the flexibility of the pool structure and water mass are considered in the building model for seismic soil-structure interaction (SSI) analysis. The calculated response of the pool structure is then used to determine the hydrodynamic response of the pool water. The specific equations used are as follows:

- Sloshing frequency, ω

  $$\omega^2 = \frac{1.58g}{l}\tanh\left(1.58\frac{h}{l}\right)$$

  in which l is half length of pool, h is effective depth of pool, and g is acceleration of gravity.

- Sloshing pressure on walls, p_s

  $$p_s = \left(\frac{\gamma}{g}\right)\cdot\frac{l^2}{3}\cdot\sqrt{\frac{5}{2}}\cdot\frac{\cosh\left(\sqrt{\frac{5}{2}}\cdot\frac{h-y}{l}\right)}{\sinh\left(\sqrt{\frac{5}{2}}\cdot\frac{h}{l}\right)}\cdot\omega^2\cdot\theta_h$$

  $$\theta_h = 1.58\frac{A_1}{l}\tanh\left(1.58\frac{h}{l}\right)$$

  in which γ is water density, y is the distance measured from the pool surface, and A_l is the 0.5% damped horizontal spectral displacement at the sloshing frequency.

- Impulsive pressure on walls, p_w

  $$p_w = \gamma\left(\frac{A_{\max}}{g}\right)\cdot h\cdot\left[\frac{y}{h}-\frac{1}{2}\cdot\left(\frac{y}{h}\right)^2\right]\cdot\sqrt{3}\cdot\tanh\left(\sqrt{3}\frac{l}{h}\right)$$

  in which A_{max} is the averaged maximum horizontal acceleration (in g's) over the height of the pool structure calculated from the building SSI analysis which is presented in DCD Tier 2, Section 3A.9 for site envelope seismic responses.

In addition, the hydrodynamic pressures on walls due to the vertical earthquake component considered in the design are determined as:

- $p_v = A_v \cdot y \cdot \gamma$

  in which A_v is the averaged maximum vertical acceleration (in g's) over of the height of the pool structure calculated from the building SSI analysis, which is presented in DCD Tier 2, Section 3A.9 for site envelope seismic responses.

Consider Gravity-Driven Cooling System (GDCS) Pool A as an example. The computed sloshing frequency is 0.205 Hz and the sloshing pressure on the reinforced concrete containment vessel (RCCV) wall at a depth of 5.4m is 4.95 kN/m^2. The impulsive pressure at this location is 51.53 kN/m^2. The total pressure obtained by the SRSS method is 51.77 kN/m^2. Thus, the impulsive response is over 99% of the horizontal seismic-induced hydrodynamic load for this pool and at this elevation in the N-S direction. All locations were reviewed and it was found that the impulsive response ranges from 82.1% to 99.9% of the horizontal seismic-induced hydrodynamic loads, with an average value of 98.2%. The contribution of sloshing pressure to the total wall pressure is reduced further when the vertical seismic-induced hydrodynamic pressure is included. It can thus be concluded that the majority of hydrodynamic loads are due to the impulsive response of the water; the sloshing response is only a small portion of overall seismic-induced hydrodynamic loads on the pool structure and does not govern.

The following ACI 350.3-01 equations for impulsive response of rectangular tanks are considered for comparison with the design method for the impulsive response described above.

- $$P_{iy} = \frac{\frac{P_i}{2}\left[4H_L - 6h_i - \left(6H_L - 12h_i\right)\left(\frac{y}{H_L}\right)\right]}{H_L^2}$$

- $$P_i = ZSIC_i \frac{W_i}{R_{wi}},$$

  Note the expression $ZSIC_i/R_{wi}$ is the maximum acceleration of the tank.

- $$\frac{W_i}{W_L} = \frac{\tanh\left[0.866\left(L/H_L\right)\right]}{0.866\left(L/H_L\right)}$$

  where,

  h_i = height above the base of the wall to the center of gravity of the impulsive lateral force.

  H_L = design depth of stored liquid.

  L = inside length of a rectangular tank, parallel to the direction of the earthquake force.

  P_i = total lateral impulsive force associated with W_i.

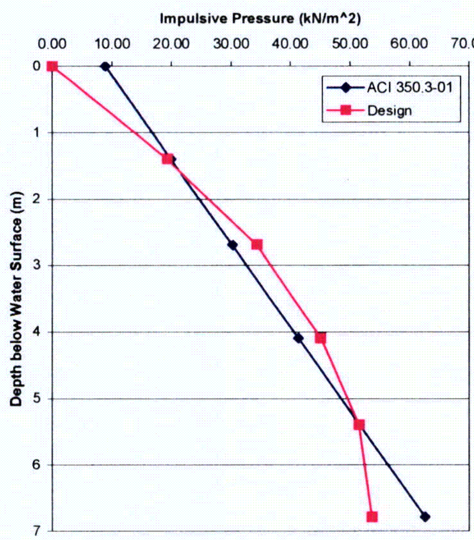
  P_{iy} = lateral impulsive force due to W_i, per unit height of the tank wall, occurring at level y above the tank base.

  W_i = equivalent mass of the impulsive component of the stored liquid.

  W_L = total mass of the stored liquid.

  y = liquid level at which the wall is being investigated (measured from tank base).

The impulsive pressures using the ACI equations compared to the design impulsive pressures are shown below for GDCS Pool A, which is not fully rectangular but was analyzed as an equivalent rectangular tank.



The two pressure distributions are reasonably close with the design pressures being higher in the middle portions of the wall and the ACI pressures being higher near the wall edges. The higher design pressures in the wall's middle span are more critical to the out-of-plane bending of the wall. The total lateral impulsive forces computed to be the area under the curve are

1251 kN and 1266 kN using the design method and ACI method, respectively. The difference is only about 1%, which is negligible.

Lastly, the statement that the impulsive response inherent in the CSDRS-based design is typically an order of magnitude higher than the sloshing response at lower accelerations of the FIRS is justified by examining FSAR Figure 2.5.2-233. This figure shows the RB/FB horizontal FIRS in comparison with the ESBWR horizontal CSDRS. At the fundamental frequency of 1.19 Hz for the RB/FB at a soft site (see response to RAI Item No. 1 above), the spectral acceleration of the CSDRS is about 0.5g, which is further amplified in the impulsive response due to multiple-mode response of the building structure. At 0.2 Hz, below which the FIRS exceeds the CSDRS, the spectral acceleration of the FIRS is about 0.09g, which is related to the sloshing response independent of building frequencies. The ratio of CSDRS acceleration at 1.19 Hz to the FIRS acceleration at 0.2 Hz is about 5.5. The ratio is larger when multi-mode response of the building is taken into account. The CSDRS-based building response acceleration considered for the impulsive mode of GDCS Pool A is 0.96g. The ratio to FIRS of 0.09g for the sloshing mode is larger than 10.

RAI Item No. 3:

The sloshing frequency of the firewater storage tank is calculated to be 0.24 Hz using ASCE 4-98, Equation C3.5-9,

$$f = \frac{\omega_2}{2\pi} = \frac{1}{2\pi}\sqrt{\frac{3.67g}{D}\tanh\left(\frac{3.67H}{D}\right)}$$

in which D is the tank inside diameter, H is the fluid height and g is acceleration of gravity.

The same equation, but arranged in a different expression, is used in ACI 350.3-01. The sloshing mode is represented by a single mass-spring system as node 60 in the FWSC seismic model shown in DCD Tier 2, Figure 3A.7-7. The sloshing frequency calculated by the eigenvalue analysis is confirmed to be 0.24 Hz in each of two horizontal directions as shown for the first two modes in DCD Tier 2, Tables 3A.7-15 through 3A.7-18, for all site conditions.

RAI Item No. 4:

There are no circular tanks included as an integral part of the building structures in the ESBWR RB/FB standard design. One side of the GDCS pools is part of the reinforced concrete cylindrical containment structure for which buckling potential is excluded by meeting the allowable compressive stress in concrete and tensile stress in reinforcing steel. There are, however, some stand-alone steel circular tanks, such as the backwash and Standby Liquid Control (SLC) tanks, housed in the RB/FB. These tanks are procured components designed in accordance with ASME or API Code to ensure no buckling failure under the CSDRS-based seismic loads including the vertical earthquake component-induced axisymmetric hydrodynamic pressure. The period of vibration in the vertical direction, T_v, for circular tanks can be computed from

$$T_v = 2\pi\sqrt{\frac{\gamma_L D H_L^2}{2 g t_w E}}$$

in which γ_L is specific weight of contained liquid, t_w is average wall thickness and E is Young's modulus of tank material. Other notations are the same as in the responses to RAI Items No. 2 and No. 3, above. This is Equation 4-17 in ACI 350.3-01 and is also applicable to steel circular tanks. For typical backwash and SLC tanks used in nuclear plant applications, the vertical frequencies are in the range of 30 Hz and above, much higher than 0.15 Hz, below which the vertical FIRS exceeds CSDRS.

Proposed COLA Revision

None

ATTACHMENT 2

G3NO-2008-00021

RESPONSE TO NRC RAI LETTER NO. 17

RAI QUESTION NO. 03.07.01-2

RAI QUESTION NO. 03.07.01-2

NRC RAI 03.07.01-2

QUESTION for Structural Engineering Branch 2 (ESBWR/ABWR Projects) (SEB2)

With respect to the applicant's assertion provided in the GGNS Unit 3 COL Application FSAR Section 3.7.1.1.5 that: "Approach 3 of NUREG/CR-6728 was used to develop FIRS at the various foundation levels and did not require the use of acceleration time history," it should be noted that the acceptability of NUREG/CR-6728 and that of Enclosure 1 to Entergy's June 30, 2008 letter titled, "Supplemental Information Regarding Methodology Used to Develop Horizontal and Vertical Site-Specific Hazard Consistent Uniform Hazard Response Spectra," are still under review by the staff, and staff acceptance of the above applicant's assertion is predicated on a satisfactory completion of the staff review.

The staff requests the applicant to discuss if the Random Vibration Theory (RVT) approach was used in conjunction with the Approach 3 in generating the site specific FIRS shown in GGNS Unit 3 COL Application FSAR Figures 2.0-201 and 2.0-202.

Entergy Response

The Random Vibration Theory (RVT) approach was used in conjunction with Approach 3 to generate the site-specific Foundation Input Response Spectra (FIRS) shown in GGNS Unit 3 COL Application (FSAR Figures 2.5.2-235, 2.5.2-236, and 2.5.2-237), as discussed in FSAR Sections 2.5.2.4.2 and 2.5.2.4.2.1. Strain-compatible analyses were conducted using the RVT approach (FSAR Sections 2.5.2.3 and 2.5.2.4.2.1.1, and FSAR Table 2.5.2-207) consistent with RG 1.208 guidance (Page 16, last sentence), which states, "RVT methods are acceptable as long as the strain dependent soil properties are adequately accounted for in the analysis." Additional background and implementation detail on the use of RVT for site response analyses, as part of Approach 3 implementation, is provided in Section 2.0 of Enclosure 1 to Entergy's June 30, 2008 letter (CNRO-2008-00020)[1] to the NRC regarding the use of Approach 3.

Proposed COLA Revision

FSAR Section 2.5.2.4.2.1 will be revised as indicated in the attached draft markup to clarify the usage of the RVT method.

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[1] ADAMS Accession No. ML081840285

Markup of Grand Gulf COLA

The following markup represents Entergy's good faith effort to show how the COLA will be revised in a future COLA submittal in response to the subject RAI. However, the same COLA content may be impacted by revisions to the ESBWR DCD, responses to other COLA RAIs, other COLA changes, plant design changes, editorial or typographical corrections, etc. As a result, the final COLA content that appears in a future submittal may be somewhat different than as presented herein.

site-specific horizontal and vertical hazard curves reflecting site aleatory and epistemic variabilities.

- Computation of site-specific UHRS and DRS.

2.5.2.4.2.1 Development of Transfer Functions

Transfer functions include spectral ratios (5% damping) of horizontal soil motions to hard rock (Table 2.5.2-208) as well as vertical-to-horizontal ratios (5% damping) computed for the site-specific soil profiles for a suite of expected peak accelerations (0.01 to 1.50 g; Table 2.5.2-208).

To approximate nonlinear soil response for horizontal motions, an RVT based equivalent-linear approach was used that accounted for strain-dependent soil properties (Reference 2.5.2-227). The approach has been validated by modeling strong ground motions recorded at over 500 sites and 19 earthquakes for a wide range in site conditions and loading levels (up to 1g) (Reference 2.5.2-217 and Reference 2.5.2-227). Comparisons with fully nonlinear codes for loading levels up to 1g showed the equivalent-linear approach adequately captured both high- and low-frequency soil response in terms of 5% damped response spectra. The validations revealed that the equivalent-linear approach significantly underestimated durations (time domain) of high-frequency motions at high loading levels compared to both fully nonlinear analysis as well as recorded motions. However, for 5% damped response spectra the equivalent-linear approach performed as well as fully nonlinear codes and was somewhat conservative near the fundamental column resonance (Reference 2.5.2-217). For vertical motions, site-specific V/H ratio were developed using the point-source model to compute both horizontal (normally incident SH-waves) and vertical (incident inclined P- SV-waves) (Reference 2.5.2-216 and Reference 2.5.2-228).

Empirical western North America (WNA) V/H ratios were included in the development of vertical motions in addition to site-specific point-source simulations. The use of WNA empirical V/H ratios implicitly assumes similarity in shear- and compression-wave profiles as well as nonlinear dynamic material properties between deep firm soils in WNA and site-specific soil columns. While this may not be the case for the average WNA deep firm soil (Reference 2.5.2-229), the range in soil conditions sampled by the WNA empirical generic rock and soil relations likely accommodates the local Holocene and Pleistocene soils. Additionally, because the model for vertical motions is not as thoroughly validated as the model for horizontal motions (Reference 2.5.2-216, Reference 2.5.2-227, and Reference 2.5.2-229), inclusion of empirical models is warranted. The additional epistemic variability introduced by inclusion of both analytical and empirical models also appropriately reflects the difficulty and lack of industry consensus on developing (modeling) site-specific vertical motions (Reference 2.5.2-219). In the implementation of Approach 3 to develop vertical hazard curves, the epistemic variability is properly accommodated in the vertical mean UHRS, reflecting a weighted average over multiple vertical hazard curves computed for

ATTACHMENT 3

G3NO-2008-00021

REGULATORY COMMITMENTS

REGULATORY COMMITMENTS

The following table identifies those actions committed to by Entergy in this document. Any other statements in this submittal are provided for information purposes and are not considered to be regulatory commitments.

COMMITMENT	TYPE (Check one)		SCHEDULED COMPLETION DATE (If Required)
	ONE-TIME ACTION	CONTINUING COMPLIANCE	
FSAR Section 2.5.2.4.2.1 will be revised to clarify the usage of the RVT method (see Attachment 2).	✓		Future COLA submittal.