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August 28, 2012
U7-C-NINA-NRC-120057

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

Attachments 1 and 2 provide supplemental or revised responses to NRC staff questions 03.08.04-18 and 03.08.04-23 related to the Combined License Application (COLA) Part 2, Tier 2, Section 3.8. Following the audit performed during the week of July 23, 2012, the NRC Staff requested that Nuclear Innovation North America LLC provide additional information to support the review of the COLA. These submittal responses complete the actions requested by the NRC Staff.

Where there are COLA markups, they will be made at the first routine COLA update following NRC acceptance of the RAI response. There are no commitments in this letter.

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

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

Executed on 8/28/12

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

jep

Attachments:

1. RAI 03.08.04-18, Supplement 4
2. RAI 03.08.04-23, Revision 1

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NRO

STI 33588696

cc: w/o attachment except*
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RAI 03.08.04-18, Supplement 4

QUESTION:

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

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

SUPPLEMENTAL RESPONSE:

The Supplement 3 response to this RAI was submitted with Nuclear Innovation North America (NINA) letter U7-C-NINA-NRC-110103, dated July 27, 2011. This supplement provides the response to Punch List Items 261 through 267 and 272.

Each of the above 8 Punch List Items is addressed below.

1. *In Subsection 3H.6.4.3.4.2, specify that a lower stress limit of 1.4 will be used for shear* ***(Punch List Item 261)***

   See revised COLA Section 3H.6.4.3.4.2 provided in Enclosure.

2. *Include in COLA a discussion of foundation soil springs, spring values, and reason for using uniform soil springs for RWB SAP2000 model* ***(Punch List Item 262)***

   The RWB SAP2000 finite element model includes uniform foundation soil springs. The RWB foundation is very rigid because the basemat is 12 ft. thick and it is stiffened with interior shear walls arranged approximately every 30 ft. in both the east-west and the north-south directions. Therefore, no significant dishing of the mat is expected and the use of uniform foundation soil springs is appropriate. The static subgrade reaction modulus for the vertical springs is 50 kips/ft/ft^2. The dynamic subgrade reaction modulus for the vertical springs is 184 kips/ft/ft^2.

   See enclosed revised COLA Section 3H.3.5.2.

3. *Include the basis for selection of anchor bolt material for RWB* ***(Punch List Item 263)***

The anchor bolt material specified for the RWB is ASTM F1554. This material is the preferred anchor bolt material endorsed by ANSI/AISC N690-12.

See enclosed revised COLA Sections 3H.3.4.4.5 and 3H.6.4.4.5.

4. *Provide basis for using no load for wall attachments in the structural design in an RAI response* ***(Punch List Item 264)***

Due to conservative design, small number of wall attachments, and required prior approval for wall attachments during detailed design and to minimize excessive over-design no wall attachment loads were assumed for the basic design of UHS/RSW Pump House, Radwaste Building, RSW Piping Tunnels and Diesel Generator Fuel Oil Storage Vaults (DGFOSV). Wall attachment loads were considered in the basic design of Diesel Generator Fuel Oil Tunnels (DGFOT). See below for additional information.

Controls During Detailed Design:

Prior to completion of detailed design, it is not possible to know all attachment loads and locations. Thus, in the basic design stage, based on the available information, some attachment loads are assumed knowing that such assumptions will not be violated due to controls that will be in place during the detailed design.

During the detailed design, any attachment to the structure (i.e. walls, slabs, etc.) requires prior approval of the structural engineering. The approval will be granted only if one of the following is met:

- The attachment loads are bounded by the attachment loads considered in the basic design
- The affected structural elements can accommodate the attachment loads without any overstress
- Additional local reinforcement is provided to accommodate the attachment loads

Small Number of Wall Attachments:

As described below, few wall attachments are anticipated for the subject structures.

UHS/RSW Pump House:
Possible wall attachments will be in the RSW Pump House, however, very few wall attachments are anticipated.

Radwaste Building:
Few wall attachments are anticipated and they mostly will be located below grade.

RSW Piping Tunnels:
RSW Piping will be supported off the tunnel floor slabs. Cable trays will be supported off the ceilings. Attachment loads are considered for tunnel floors and ceilings.

DGFOSV:
No significant wall attachments are anticipated.

DGFOT:
Fuel oil piping will be supported off the walls, thus wall attachment loads are considered.

Conservative Design:

Design of UHS/RSW Pump House, Radwaste Building, DGFOSV and DGFOT were carried out using finite element models. These designs are conservative since the design is an element based design. Therefore, unless the walls are subjected to very significant concentrated out-of-plane loads, they will be adequate without requiring any additional local reinforcement.

Considering the above, the designs of the UHS/RSW Pump House, Radwaste Building, RSW Piping Tunnels and DGFOSV structures with no assumed wall attachment loads are adequate because during the detailed design the wall attachments for these structures will require prior approval of the structural engineering.

5. *Provide additional explanation for simultaneous consideration of sliding and overturning about two horizontal axes in COLA* ***(Punch List Item 265)***

As noted in COLA Sections 3H.6.5.2.14, 3H.6.7 and 3H.7.5.3.4, in the stability evaluations the 100%, 40%, 40% rule is used for combining the three X, Y, and Z seismic components. COLA Figure 3H.6-137 presents the formulations for sliding and overturning check for a single horizontal direction earthquake. When considering two horizontal (X and Y) excitations, the formulations of Figure 3H.6-137 remain unchanged except that the friction force (F) along the X or Y direction is replaced with Fx and Fy (friction force along the x and y axes, respectively). Fx and Fy forces are determined as follows:

Let:

Rx = Total driving sliding force along the x-axis
Ry = Total driving sliding force along the y-axis
R = Resultant driving sliding force = $[Rx^2 + Ry^2]^{1/2}$
F = Total friction force as defined in Figure 3H.6-137
Fx = Friction force along the x-axis
Fy = Friction force along the y-axis

Then,

Fx = F(Rx/R)
Fy = F(Ry/R)

See enclosed revised COLA Sections 3H.6.5.2.14, 3H.6.7 and 3H.7.5.3.4.

6. *Revise COLA Figure 3H.6-137 to remove reference to UHS figures* ***(Punch List Item 266)***

   See Enclosure for COLA markup of Figure 3H.6-137.

7. *Revise COLA Figures 3H.6-48 through 50 to show passive soil pressures needed for stability* ***(Punch List Item 267)***

   See Enclosure for COLA markup of Figures 3H.6-48 through 3H.6-50.

8. *Revise the construction sequence COLA mark up as follows:*
   a) *For UHS/RSW Pump House, add an additional bullet to address construction above UHS basemat level*
   b) *For buried tunnels, revise first bullet to clarify that construction should be uniform and level by level*

   (Punch List Item 272)

   COLA markups for Section 3.8.5.10 have been updated to include the above two items.

See Enclosure for COLA markups.

Enclosure

COLA MARKUPS

3.8.5.10 Construction Sequencing for Seismic Category I Foundations

In order to assure that construction loading does not result in excessive stresses on the foundation mat or the superstructure, construction sequence planning will consider the following:

- Construction should proceed such that major walls at the lowest level, those providing foundation mat stiffness, are constructed across essentially the entire foundation before loads from walls and slabs above are applied
- Loads should be uniformly applied to the foundations
- Overall foundation tilt would remain within 1/600.

Construction specifications will include the following requirements:

- The concrete placement for the superstructure will be such that the superstructure is erected uniformly considering the following:
    - Concrete pours for major walls are limited to the lesser of about 20'-0" or to the floor above until all of the major walls at that elevation are poured.
    - Concrete pours for major floor slabs are essentially completed for the entire floor before concrete pours are started for floor above.
- For the RSW Pump House/UHS foundation, the following sequence will be specified:
    - Excavate the RSW Pump House/UHS to the bottom of the UHS foundation
    - Place the UHS foundation concrete to a construction joint within about ten feet of the junction with the RSW Pump House
    - Drive sheet piling along the RSW Pump House wall adjacent to the UHS and excavate to the bottom of the RSW Pump House foundation
    - Place the RSW Pump House foundation concrete
    - Place the RSW Pump House concrete walls up to the UHS foundation level
    - Complete concrete placements for UHS foundations and RSW Pump House slabs at the top of the UHS foundation level
    - For the remaining portions of the UHS basin and RSW Pump House above the UHS basemat level the concrete pour of the major walls will be limited to the lesser of about 20'-0" or to the floor above until all of the major walls at that elevation are poured.
- For the buried tunnels, the following sequence will be specified:
    - Construct the tunnels uniformly and level by level to a construction joint within about ten feet of the junction with the terminating structure
    - After placing backfill around and above each tunnel, place the last tunnel segment adjacent to the terminating structure

3H.3.4.4.5 Anchor Bolts

Material for anchor bolts conforms to the requirements of ASTM F1554 (preferred anchor bolt material endorsed by ANSI/AISC N690-12), Grade 36. Its design properties are:

- Yield strength .... 36 ksi (248 MPa)
- Tensile strength .... 58 ksi (400 MPa)

3H.3.5.2 Analysis and Design

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

The RWB SAP2000 finite element model includes uniform foundation soil springs. The RWB basemat is 12 ft. thick and it is stiffened with interior shear walls arranged approximately every 30 ft. in both the east-west and the north-south directions. Therefore, no significant dishing of the mat is expected and the use of uniform foundation soil springs is appropriate. The static subgrade reaction modulus for the vertical springs is 50 kips/ft/ft^2. The dynamic subgrade reaction modulus for the vertical springs is 184 kips/ft/ft^2.

Per Table 1 of RG 1.143 Revision 2, all concrete and steel designs are in accordance with the ACI 349-97 and ANSI/AISC N690, 1984 code requirements, respectively.

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

The structural steel member sizes, critical forces, safety margins, and governing load combinations for the operating floor beams, roof truss members, and roof purlins are shown in Table 3H.3-5. The layout of the operating floor steel beams is shown in Figures 3H.3-43 through 3H.3-46. The layout of the roof truss members and roof purlins are shown in Figure 3H.3-47. The typical east-west spanning truss and typical north-south spanning truss are shown in Figures 3H.3-48 and 3H.3-49, respectively.

3H.6.4.3.4.2 Structural Steel Load Combinations

S = $D + L + H + F + R_o + T_o$

S = $D + L + W + R_o + H + F + T_o$

1.6S (Note 1) = $D + L + Wt + H + R_o + F + T_o$

1.6S (Note 1) = $D + L + FL + H + R_o + F + T_o$

1.6S (Note 1) = $D + L + E' + H' + R_o + F + T_o$

1.6S (Note 1) = $D + L + S_E + R_o + H + F + T_o$

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

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

3H.6.4.4.5 Anchor Bolts

Material for anchor bolts conforms to the requirements of ASTM F1554 (preferred anchor bolt material endorsed by ANSI/AISC N690-12), Grade 36. Its design properties are:

- Yield strength .... 36 ksi (248 MPa)
- Tensile strength .... 58 ksi (400 MPa)

3H.6.5.2.14 Determination of Seismic Overturning Moments and Sliding Forces for Seismic Category I Structures

The evaluation of seismic overturning moments and sliding accounts for the simultaneous application of seismic forces in three directions using 100%, 40%, 40% combination rule as shown below:

±100% X-excitation ±40% Y-excitation +40% Z-excitation
±40% X-excitation ±100% Y-excitation +40% Z-excitation

(Note: X & Y are horizontal axes and Z is vertical axis. Positive Z is upward. Also, ±40% X-excitation ±40% Y-excitation ±100% Z-excitation is not critical for the UHS/RSW Pump House).

The resisting forces and moments due to dead load are calculated using a reduction factor of 0.90. Resisting forces and moments due to soil are based on at-rest soil pressure, or passive soil pressure, as appropriate. The friction coefficients used for the sliding evaluation are 0.30 under the RSW Pump House and 0.40 under the UHS Basin. See Figure 3H.6-137 for formulations used for calculation of factors of safety against sliding and overturning. The calculated stability safety factors for the UHS/RSW Pump

House are provided in Table 3H.6-5.

Note: Figure 3H.6-137 presents the formulations for sliding and overturning check for a single horizontal direction earthquake. When considering two horizontal (X and Y) excitations, the formulations of Figure 3H.6-137 remain unchanged except that the friction force (F) along the X or Y direction is replaced with Fx and Fy (friction force along the x and y axes, respectively). Fx and Fy forces are determined as follows:

Let:

Rx = Total driving sliding force along the x-axis
Ry = Total driving sliding force along the y-axis
R = Resultant driving sliding force = $[Rx^2 + Ry^2]^{1/2}$
F = Total friction force as defined in Figure 3H.6-137
Fx = Friction force along the x-axis
Fy = Friction force along the y-axis

Then,

Fx = F(Rx/R)
Fy = F(Ry/R)

3H.6.7 Diesel Generator Fuel Oil Storage Vaults (DGFOSV)

Stability evaluations were performed for sliding, overturning, and flotation. These evaluations were done using the procedure described in detail in Section 3H.6.5.2.14. For sliding and overturning evaluations, the 100%, 40%, 40% rule was used for consideration of the X, Y, and Z seismic excitations. Since the orientation of the DGFOSVs in the horizontal plane can be along the East-West or North-South axes, the horizontal seismic values used in the stability calculation envelope the SSI accelerations in the X and Y directions. The calculated factors of safety against sliding, overturning, and flotation for the DGFOSV are included in Table 3H.6-12.

3H.7.5.3.4 Stability Evaluation

The DGFOT stability evaluations are performed for the various load combination listed in Section 3H.7.4.5. These evaluations were done using the procedure described in detail in Section 3H.6.5.2.14. The DGFOT factors of safety against sliding, overturning, and flotation are provided in Table 3H.7-2. For sliding and overturning evaluations, the 100%, 40%, 40% rule was used for combination of the X, Y, and Z seismic excitations.

Restraints are provided around the Access Regions to limit movement and rotation due to a tornado missile.

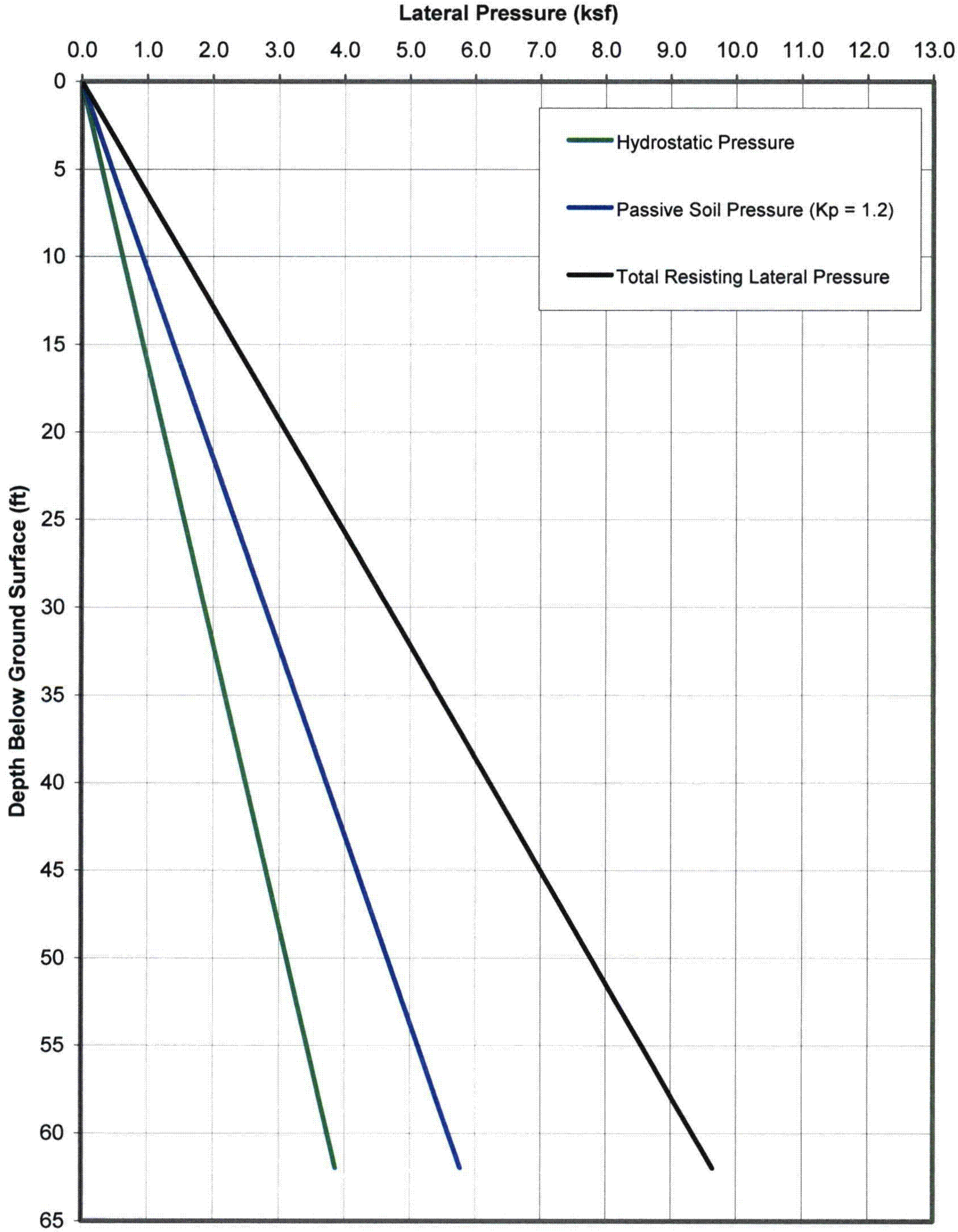


Figure 3H.6-48: Resisting Lateral Pressure on the East, West, and North Walls of Pump House (for Stability Evaluation)

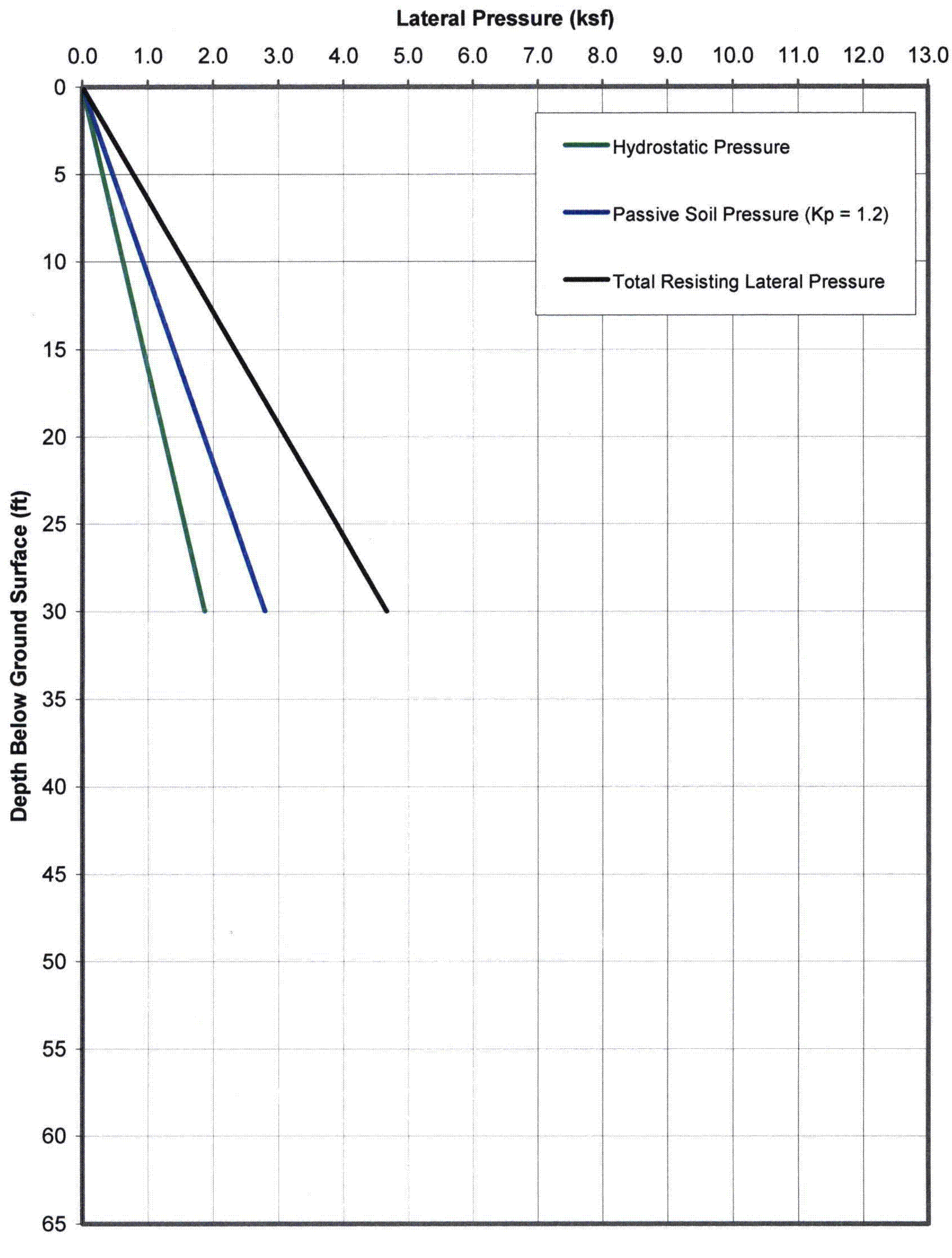


Figure 3H.6-49: Resisting Lateral Pressure on Basin Walls (for Stability Evaluation)

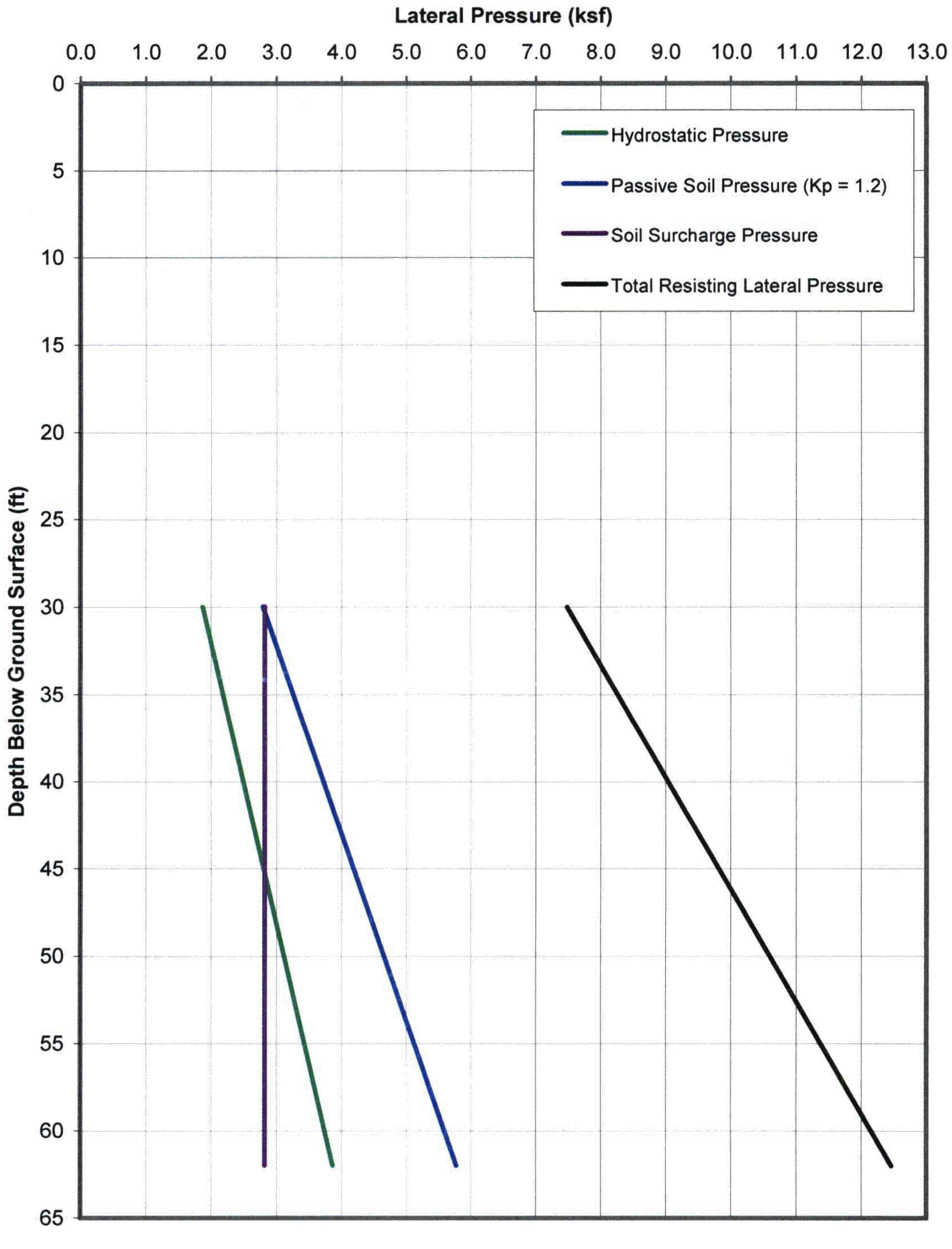
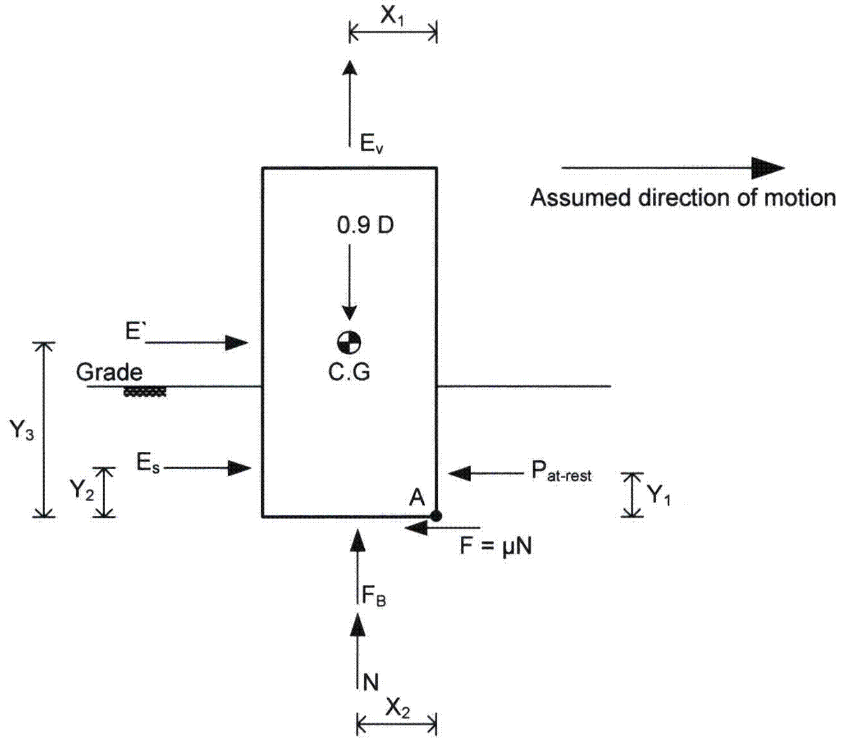


Figure 3H.6-50: Resisting Lateral Pressure on the South Wall of Pump House (for Stability Evaluation)



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

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

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

Where:

$SF_{sliding}$ = Safety factor against sliding

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

D = Dead load

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

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

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

$E`$ = Self weight excitation in the horizontal direction

E_v = Self weight excitation in the vertical direction

F_B = Buoyancy force

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

Notes:

(1) If passive pressure is utilized, $P_{passive}$ is used instead of $P_{at\text{-}rest}$

(2) E' represents the inertia of the structure and it is either determined from equivalent static method or response spectrum analysis.

(3) E_s represents the static and dynamic loads from soil which includes seismic loads from soil and hydrodynamic pressure from groundwater. These loads are computed in accordance with Section 2.5S4.10.5.

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

RAI 03.08.04-23, Revision 1

QUESTION:

Follow-up to Question 03.08.04-13 (RAI 2965)

In its response to Question 03.08.04-13, the applicant referred to FSAR mark-up provided in response to question 03.07.01-13 for structural analysis and design information for site-specific seismic category I structures (Letter U7-C-STP-NRC-090112 dated August 20, 2009). The staff noted that the above referenced response did not include all tables and figures referenced in the FSAR mark-up, and these are stated to be provided later. In addition, the level of detail included in FSAR Section 3H.6.6.3 regarding structural design of the various elements of site-specific structures is not sufficiently descriptive, and is not similar to that included in the ABWR DCD. Therefore, the applicant is requested to include in FSAR Section 3H.6.6.3 description of the various steel and concrete elements of the site specific structures including how these elements are designed including design results.

REVISED RESPONSE:

The original response to this RAI was submitted with STPNOC letter U7-C-STP-NRC-100036, dated February 10, 2010. This revision is being provided in response to Punch List Item 268 to explain why lower bound soil spring values were selected for use in the Finite Element Analysis (FEA) of Ultimate Heat Sink (UHS)/Reactor Service Water (RSW) pump house. The revisions are indicated by revision bars in the margin.

The Supplement 2 response to RAI 03.07.01-13 (see letter U7-C-STP-NRC-090230, dated December 30, 2009) contains the tables and figures that provide the design summary for the structural design of Ultimate Heat Sink (UHS) basin, UHS cooling tower enclosures, Reactor Service Water (RSW) pump house, and the RSW piping tunnels. The Supplement 2 provided the following:

- Table 3H.6-5: Factors of safety against sliding, overturning and flotation for UHS/RSW Pump House
- Table 3H.6-6: Results of RSW Piping Tunnel Design
- Table 3H.6-7: Results of UHS/RSW Pump House Concrete Wall Design
- Table 3H.6-8: Results of UHS/RSW Pump House Concrete Slab Design
- Table 3H.6-9: Results of UHS/RSW Pump House Beams and Column Design
- Table 3H.6-10: Tornado Missile Impact Evaluation for UHS/RSW Pump House

- Figures 3H.6-41 through 3H.6-43: At-rest lateral soil pressure diagrams for design of UHS/RSW Pump House
- Figure 3H.6- 44: At-rest lateral soil pressure diagram for design of RSW Piping Tunnels
- Figures 3H.6-45 through 3H.6-50: Lateral soil pressure diagrams used for stability evaluation of UHS/RSW Pump House

- Figures 3H.6-51 through 3H.6-136: Definition of reinforcement zones for UHS/RSW Pump House walls and Slabs

A three dimensional Finite Element Analysis (FEA) as shown in Figure 3H.6-40, provided with Supplement 1 Response to RAI 03.07.01-13 (see STPNOC letter U7-C-STP-NRC-090208 dated November 19, 2009), is used for structural analysis and design of the UHS/RSW Pump House.

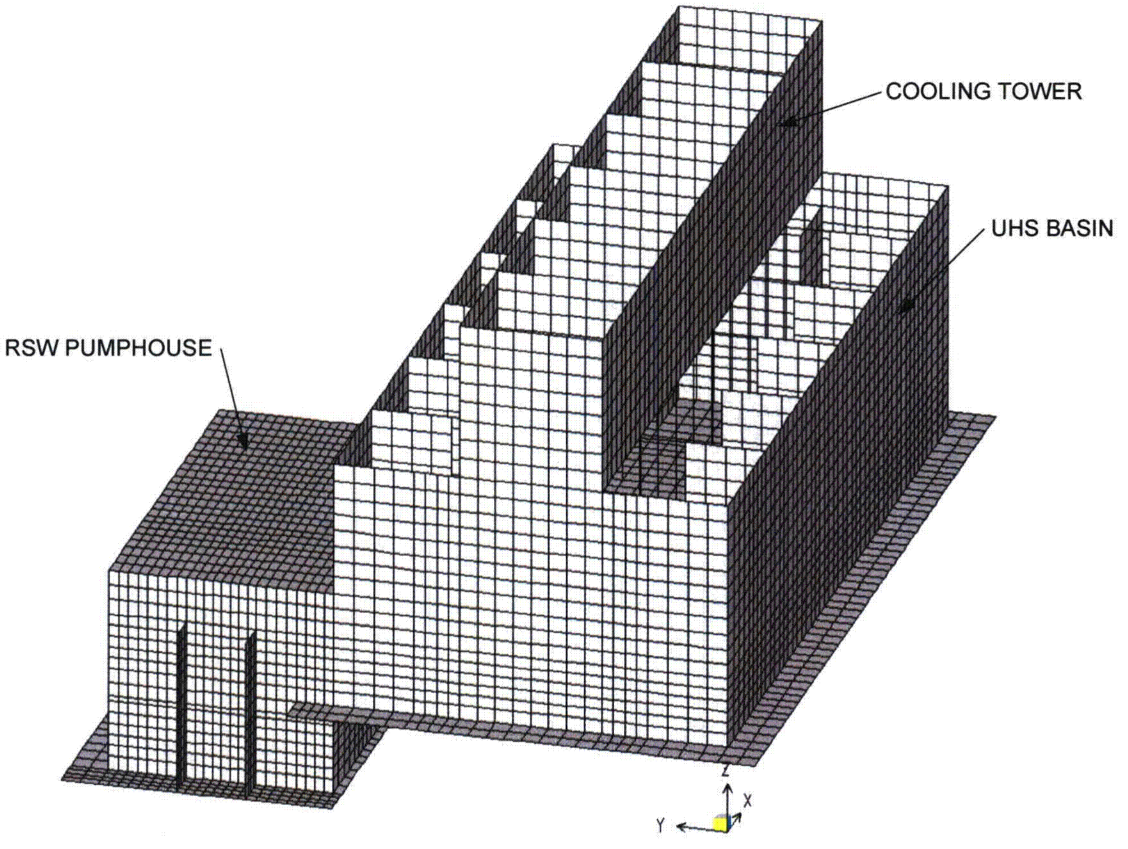


Figure 3H.6-40: SAP Finite Element Model for UHS and RSW Pump House Design

The forces in the structure caused by differential settlements due to the flexibility of the basin and pump house supporting soil are accounted for through the use of foundation soil springs in the FEA model. The methodology for computing the soil springs is presented next, followed by the values obtained.

<u>Soil Springs – Static Loading</u>

The calculated settlements due to the loading of the individual structures (S_{ss} in COLA Part 2, Tier 2, Table 2.5S.4-42) are the relevant quantity for calculating the soil spring under static loading. The unit static spring in units of force / length^3 is determined using the following equation from Section 10.5 of Bowles 1996, COLA Part 2, Tier 2, Reference 2.5S4-55 (Reference 1):

$k = q_c / S_{ss}$ force / length^3
q_c = applied foundation stress
S_{ss} = settlement of structure only due to q_c
k = modulus of subgrade reaction

As described by Bowles 1996, the modulus of subgrade reaction is an average of several local values within the foundation area. Table 2.5S.4-42 provides settlement (s_{ss}) values at nine locations in various building foundations. The local modulus of subgrade reaction (k) value at each of these 9 locations was computed and the average k for the foundation was computed as the average of these 9 local values.

Soil Springs – Seismic Loading

Reference 2 (Gazetas, 1991) provides algebraic formulas for computing the spring constants of foundations supported on/in a homogeneous half-space. These foundations have a rigid basemat of any realistic solid geometric shape. The embedded foundations are prismatic, having a sidewall-soil contact surface of height *d,* which may be a fraction of the total embedment depth *D.*

The algebraic equations of Reference 2 (Gazetas, 1991) were used to compute the soil springs for seismic loading. The algebraic equations to calculate the spring constant of a foundation require a single value of soil modulus (and Poisson's ratio) as input. The soil at the STP 3&4 consists of multiple layers, each with a shear modulus specific to the layer. Therefore, use of the equations for a homogeneous half-space requires finding a way to determine an appropriate value of shear modulus that accounts for the presence of the multiple soil layers. Reference 3 (Christiano, et. al., 1974) presents a method for obtaining the equivalent stiffness coefficients for a foundation resting on a layered system such as at the STP 3&4. The equations in Reference 3 (Christiano, et. al., 1974) can be used to calculate the appropriate single value for the soil modulus that represents the contribution of the soil layers within the influence zone of the foundation. The method involves weighting the contribution of each layer in proportion to its elastic modulus and its depth below the foundation. This weighting is done using the concept of strain energy occurring in each layer. The foundation area is represented as an equivalent circular shape and the cumulative strain energy is plotted against a dimensionless depth ratio: depth/radius. The strain energies are plotted in Reference 3 (Christiano, et. al. 1974) for vertical, horizontal, rocking and twisting displacements of the foundation. Only the vertical and horizontal modes are considered herein. The cumulative strain energy plot for the vertical mode shows an influence zone depth of 10 times the radius of the equivalent circular area, or 5 times the diameter (width). The plot for horizontal mode shows an influence zone depth of 5 times the radius of the circular area, or 2.5 times the diameter (width). Thus layers even at a considerable depth contribute to the foundation stiffness.

The SSE strain-compatible shear wave velocity is used to determine the low range, best estimate and upper range of soil shear modulus, *G,* of the individual layers via the following equation:

$$G = \rho \cdot V_s$$

Where:

G = shear modulus of individual layers
V_s = shear wave velocity (S-Wave Vel.);
ρ = mass density = γ/g;
γ = unit weight; and
g = gravitational acceleration constant (32.2 ft/s^2).

From the above information, Young's modulus of elasticity, *E*, may be calculated by:

$$E = 2G \cdot (1 + \nu)$$

Where:

E = Young's Modulus of Elasticity
ν = Poisson's Ratio

Equivalent Shear Modulus of Soil-Vertical Mode

Reference 3 (Christiano, et. al, 1974) is used to compute the equivalent modulus of the layered soil under the foundation. In this procedure, an appropriate average of the shear modulus is developed whereby each layer is weighted in accordance with the strain energy in that layer. Christiano, et. al. calculate the vertical spring using Equation 8 in the Reference 3 and their chart reproduced herein as Figure 03.08.04-23a:

$$k_v = \left[\sum \frac{(1 - \nu_i)^2}{8 \cdot a \cdot \mu_i} \cdot \Delta U_i \right]^{-1} \qquad \text{(Christiano, et. al., Equation 8)}$$

Where:

k_v = the vertical stiffness of the rigid foundation;
a = the radius of the equivalent circular area of the foundation;
ν_i = Poisson's ratio of the ith layer;
μ_i = the shear modulus of the ith layer (same as G);
ΔU_i = the strain energy coefficient change over the thickness of the ith layer (difference in U values between the top and bottom of the layer as determined from Figure 03.08.04-23a).

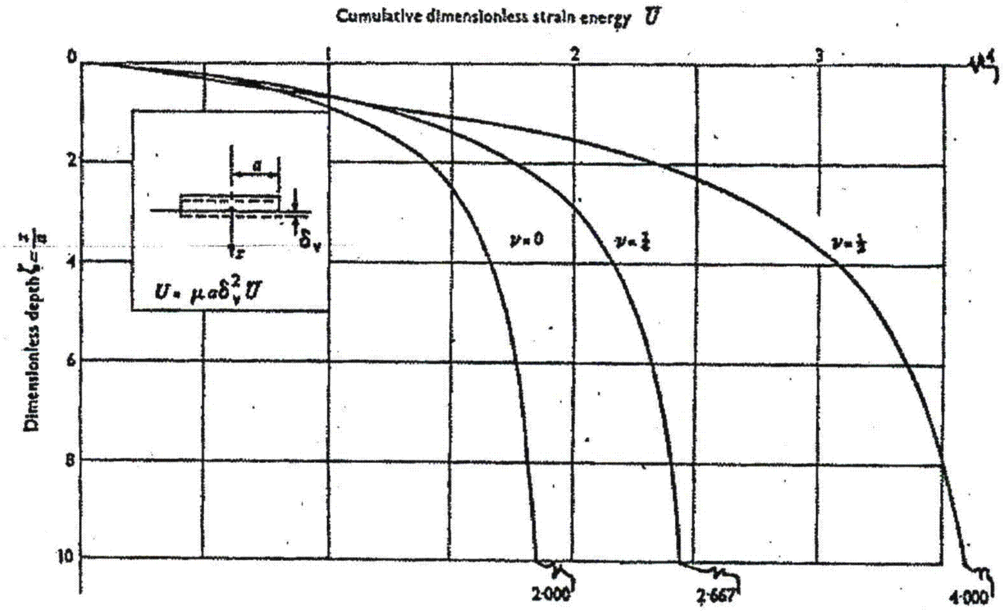


Fig. 1. Cumulative strain energy plotted against depth : vertical mode

Figure 03.08.04-23a – Cumulative Strain Energy versus Depth, Vertical Mode (Reference 3 (Christiano, et al., 1974))

For Poisson's ratios of layers intermediate between those in Figure 03.08.04-23a, linear interpolation is used.

The Poisson's ratio values (ν) for individual layers were computed from the strain-adjusted wave velocities using the following equation:

$$\nu = \frac{V_p^2 - 2V_s^2}{2(V_p^2 - V_s^2)}$$

The average Poisson's ratio, ν_{avg}, is computed as a layer-weighted value according to the following equation:

$$\nu_{avg} = \frac{\sum(\nu_i)\cdot \Delta U_i}{\sum \Delta U_i}$$

The average shear modulus (μ_{avg}) for vertical loading is back-calculated from a rearrangement of the equation in Reference 3 (Christiano, et. al. 1974) for the half space.

$$\mu_{avg} = k_v \cdot \frac{(1-\nu_{avg})}{4\cdot a}$$

Equivalent Shear Modulus of Soil-Horizontal Mode

Christiano, et al. calculate the horizontal spring using their Equation 9 in Reference 3 and their chart reproduced herein as Figure 03.08.04-23b:

$$k_h = \left[\sum \frac{(2 - v_i)^2}{32 \cdot a \cdot \mu_i} \cdot \Delta U_i \right]^{-1} \quad \text{(Christiano, et al., Equation 9)}$$

Where:

k_h = the horizontal stiffness of the rigid foundation;
a = the radius of the equivalent circular area of the foundation;
v_i = Poisson's ratio of the i^{th} layer;
μ_i= the shear modulus of the i^{th} layer (same as *G*);
ΔU_i = the strain energy coefficient change over the thickness of the i^{th} layer (difference in U values between the top and bottom of the layer as determined from Figure 03.08.04-23b).

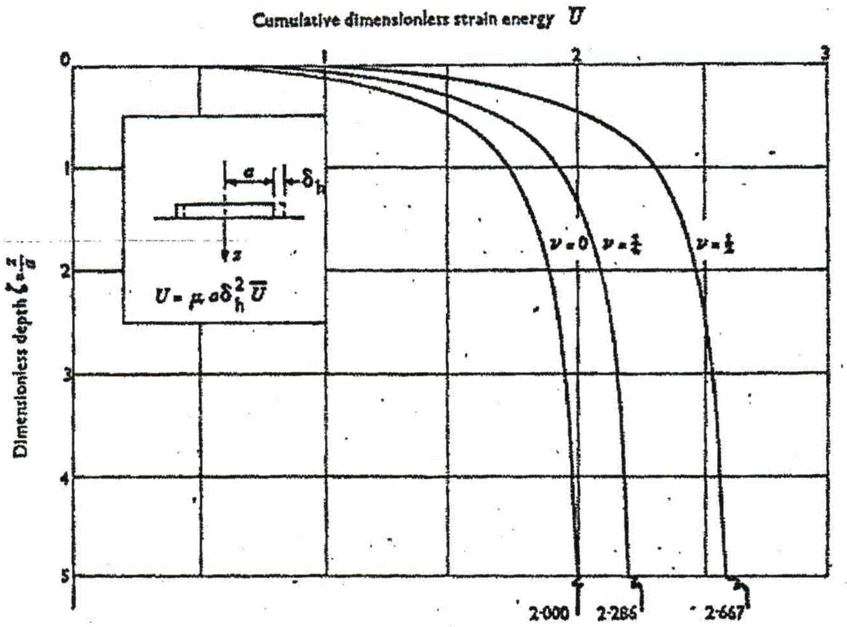


Fig. 2. Cumulative strain energy plotted against depth : horizontal mode

Figure 03.08.04-23b – Cumulative Strain Energy versus Depth, Horizontal Mode (Reference 3 (Christiano, et al., 1974))

The average shear modulus (μ_{avg}) for horizontal loading is back-calculated from a rearrangement of the equation in Reference 3 (Christiano, et al., 1974) for the half space.

$$\mu_{avg} = k_h \cdot \frac{(2 - \nu_{avg})}{8 \cdot a}$$

Gazetas Equations for Soil Seismic Springs

In the equations that follow, the shear modulus, G, is μ_{avg} and the Poisson's ratio, ν, is ν_{avg} from the preceding Christiano, et. al. equations for the vertical and horizontal modes.

Other terms in the equations are as follows:

B = ½ foundation width (parallel to y axis);
L = ½ foundation length (parallel to x axis);
h = depth to center of constant effective sidewall height;
d = constant effective sidewall height;
A_w = sidewall soil contact area, e.g. d· perimeter = d·2(2B+2L)
A_b = base soil contact area, e.g. (2B)·(2L)

$$\chi = \frac{A_b}{4L^2}$$

Vertical (z) on surface:

$$K_{z\_surf} = \frac{2GL}{1-\nu}\left(0.73 + 1.54\chi^{0.75}\right)$$

Vertical (z) embedded below surface:

$$K_{z\_emb} = K_{z\_surf}\left[1 + \frac{1}{21}\frac{D}{B}(1 + 1.3\chi)\right]\left[1 + 0.2\left(\frac{A_w}{A_b}\right)^{2/3}\right]$$

Horizontal (y) on surface

$$K_{y\_surf} = \frac{2GL}{2-\nu}\left(2 + 2.5\chi^{0.85}\right)$$

Horizontal (y) embedded below surface:

$$K_{y\_emb} = K_{y\_surf}\left(1 + 0.15\sqrt{\frac{D}{B}}\right)\left[1 + 0.52\left(\frac{h}{B}\frac{A_w}{L^2}\right)^{0.4}\right]$$

Horizontal (x) on surface

$$K_{x\_surf} = K_{y\_surf} - \frac{0.2}{0.75-\nu} GL\left(1-\frac{B}{L}\right)$$

Horizontal (x) embedded below surface

$$K_{x\_emb} = K_{x\_surf} \frac{K_{y\_emb}}{K_{y\_surf}}$$

Unit Seismic Springs

The preceding spring values have units of force/length. The springs are divided by the base soil contact area to produce unit area spring values having units of force/length^3. These springs are a composite of soil layer influences to significant depths and thus are representative of conditions anywhere on the base area of the foundation.

The UHS basin basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads)..30 kips/ft/ft^2

Vertical springs (with seismic loads).. Lower Bound 80 kips/ft/ft^2
(Mean 121 kips/ft/ft^2, Upper Bound 182 kips/ft/ft^2)

North-south springs (with static and seismic loads) .... Lower Bound 33 kips/ft/ft^2
(Mean 50 kips/ft/ft^2, Upper Bound 77 kips/ft/ft^2)

East-west springs (with static and seismic loads) ....Lower Bound 30 kips/ft/ft^2
(Mean 46 kips/ft/ft^2,Upper Bound 70 kips/ft/ft^2)

The RSW pump house basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads)... ....60 kips/ft/ft^2

Vertical springs (with seismic loads).. Lower Bound 170 kips/ft/ft^2
(Mean 251 kips/ft/ft^2, Upper Bound 375 kips/ft/ft^2)

North-south springs (with static and seismic loads) ....Lower Bound 112 kips/ft/ft^2
(Mean 173 kips/ft/ft^2 , Upper Bound 267 kips/ft/ft^2)

East-west springs (with static and seismic loads) ....Lower Bound 104 kips/ft/ft^2
(Mean 161 kips/ft/ft^2 , Upper Bound 248 kips/ft/ft^2)

Since use of softer soil springs will result in higher basemat forces, lower bound soil springs were selected for use in the Finite Element Analysis of the UHS/RSW Pump House.

Tables 3H.6-7 through 3H.6-9, submitted with Supplement 2 response of RAI 03.07.01-13 (see STPNOC letter U7-C-STP-NRC-090230, dated December 30, 2009), include the calculated design forces and the provided reinforcement for the walls, slabs, beams, and columns of the UHS basin/UHS cooling tower/RSW pump house structures. Figures 3H.6-51 through 3H.6-136, submitted with Supplement 2 response of RAI 03.07.01-13 (see STPNOC letter U7-C-STP-NRC-090230, dated December 30, 2009) show the various wall and slab reinforcement zones used to define the provided reinforcement based on the finite element analysis results. The actual provided reinforcement, based on final rebar layout, may exceed the reported provided reinforcement, and the zones with higher reinforcement may be extended beyond their reported zone boundaries.

The UHS/RSW pump house design used an iterative approach of checking the design axial force and moment couples for every load combination from the finite element model versus ACI 349-97 axial force and moment (P&M) interaction diagrams that were calculated based on actual reinforcement bar diameters, spacings, and layers. If the design axial force and moment couple for any load combination was outside of the allowable ACI 349-97 P&M interaction curve for a given reinforcement pattern, the design axial force and moment couples for every load combination were rechecked versus the allowable ACI 349-97 P&M interaction curve for a reinforcement pattern with a higher capacity (higher area of steel). When all of the axial force and moment couples from every load combination were within the allowable ACI 349-97 P&M interaction curve for a given reinforcement pattern, the area of steel corresponding to this reinforcement pattern plus any additional required reinforcement for in-plane shear was reported in Tables 3H.6-7 and 3H.6-8 as the "provided longitudinal reinforcing".

Please see the response to RAI 03.07.02-15, items 1 through 6 for information regarding the RSW Piping Tunnels.

References: The following references are used in this RAI response:

1. FSAR Reference 2.5S.4-55 "Foundation Analysis and Design, (5th edition)," Bowles, J. E., 1996.

2. Gazetas, G., 1991. "Formulas and Charts for Impedances of Surface and Embedded Foundations", Journal of Geotechnical Engineering, Vol. 117, No. 9, pages 1363-1381.

3. Christiano, P. P., Rizzo, P. C., and Jarecki, S. J., 1974. "Compliances of Layered Elastic Systems", Proceedings of the Institute of Civil Engineers, Part 2, Vol 57, December, pages 673-683.

The following COLA changes will be made to add additional details on the design of UHS/RSW Pump House.

3H.6.6.3 Structural Design

The strength design criteria defined in ACI 349 as supplemented by RG 1.142 as well as ACI 350 (note: ACI 350 is applicable only to the exterior walls below the 71 ft maximum water level and basemat of UHS basin), was used to design the reinforced concrete elements making up the UHS basin and cooling tower enclosures as well as the RSW pump house and piping tunnels. Concrete with a compressive strength of 4.0 ksi (27.6 MPa) and reinforcing steel with a yield strength of 60 ksi (414 MPa) are considered in the design.

3H.6.6.3.1 UHS Basin/UHS Cooling Tower/RSW Pump House Concrete Wall and Slab Design

The design forces and provided reinforcement for UHS basin, UHS cooling tower, and RSW pump house walls and slabs are shown in Tables 3H.6-7 and 3H.6-8. Each face and each direction of each wall and slab has a corresponding longitudinal reinforcement zone figure. Each wall and slab also has a corresponding transverse shear reinforcement zone figure when transverse shear reinforcement is required. The reinforcement zone figures (Figures 3H.6-51 through 3H.6-136) show the various zones used to define the provided reinforcement based on the finite element analysis results. Actual provided reinforcement, based on final rebar layout, may exceed the reported provided reinforcement and the zones with higher reinforcement may be extended beyond their reported zone boundaries.

The shell forces from every element for every load combination in the finite element analysis were evaluated to determine the provided reinforcement in each reinforcement zone. For each reinforcement zone, the following out-of-plane moment and axial force couples with the corresponding load combination are reported in Tables 3H.6-7 and 3H.6-8:

- The maximum tension axial force with the corresponding moment acting simultaneously from the same load combination.
- The maximum compression axial force with the corresponding moment acting simultaneously from the same load combination.
- The maximum moment that has a corresponding axial tension acting simultaneously in the same load combination.
- The maximum moment that has a corresponding axial compression in the same load combination.

For each reinforcement zone, the following in-plane and transverse shears with the corresponding load combination are reported in Tables 3H.6-7 and 3H.6-8:

- The in-plane shear is the maximum average in-plane shear along a plane that crosses the longitudinal reinforcement zone.

- The transverse shear is the maximum average transverse shear along a plane in that transverse reinforcement zone.

The provided longitudinal reinforcing for each face and each direction is determined based on the out-of-plane moments, axial forces, and in-plane shears occurring simultaneously for every load combination.

The provided transverse shear reinforcing (as required) is determined based on the transverse shears and axial forces perpendicular to the shear plane occurring simultaneously for every load combination. The UHS basin and RSW pump house basemats were also evaluated for punching shear at critical locations under buttresses and columns.

The forces in the structure caused by differential settlements due to the flexibility of the basin and pump house basemats and supporting soil were accounted for through the use of foundation soil springs in the finite element model. The soil spring stiffness values used in the finite element model were based on the calculated soil subgrade modulus, which is a function of the foundation settlement.

The UHS basin basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads)..30 kips/ft/ft^2

Vertical springs (with seismic loads)..80 kips/ft/ft^2

North-south springs (with static and seismic loads) ....33 kips/ft/ft^2

East-west springs (with static and seismic loads) ....30 kips/ft/ft^2

The RSW pump house basemat is supported by area springs with the following uniform spring constants in the finite element model:

Vertical springs (with static loads)... ....60 kips/ft/ft^2

Vertical springs (with seismic loads).. ....170 kips/ft/ft^2

North-south springs (with static and seismic loads) ... .112 kips/ft/ft^2

East-west springs (with static and seismic loads) ... .104 kips/ft/ft^2

The RSW pump house operating floor and roof were designed with composite steel beams and concrete slabs for vertical loading. The composite beams span in the east-west direction with the concrete slab designed as spanning one-way between the composite beams. The operating floor and roof slabs also act as diaphragms to transfer lateral loads. The provided reinforcing for the operating floor and roof slabs is reported in Table 3H.6-8.

3H.6.6.3.2 UHS Basin Beam and Column Design

The beams and columns in the UHS basin were represented with frame elements in the finite element model. The frame forces for every load combination in the finite element model were evaluated to determine the provided reinforcement for each beam and column in Table 3H.6-9. For each beam and column, the following forces and the corresponding load combination are reported in Table 3H.6-9:

- The maximum axial compression force with the corresponding biaxial bending moments (M2 and M3) acting simultaneously from the same load combination.
- The maximum axial tension force with the corresponding biaxial bending moments (M2 and M3) acting simultaneously from the same load combination. Note that the columns do not have an axial tension case.
- The maximum M2 bending moment with the corresponding M3 bending moment and axial force acting simultaneously from the same load combination.
- The maximum M3 bending moment with the corresponding M2 bending moment and axial force acting simultaneously from the same load combination.
- The maximum shear V2.
- The maximum shear V3.
- The maximum torsion.

The provided longitudinal reinforcing in Table 3H.6.9 is determined based on the axial force, biaxial moments (M2 and M3), and torsion. The provided stirrup reinforcing is determined based on the axial force, shears (V2 and V3), and torsion.