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UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

June 12, 2009

UN#09-291

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

Subject: UniStar Nuclear Energy, NRC Docket No. 52-016
Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3,
RAI No. 58, Seismic Design Parameters
RAI No. 63, Seismic Subsystem Analysis
RAI No. 65, Seismic System Analysis, and
RAI No. 112, Seismic Design Parameters

References:
1) John Rycyna (NRC) to Robert Poche (UniStar Nuclear Energy), "RAI No. 58 SEB2 1966.doc (PUBLIC)" email dated February 17, 2009

2) John Rycyna (NRC) to Robert Poche (UniStar Nuclear Energy), "RAI No. 63 SEB2 1973.doc (PUBLIC)" email dated February 18, 2009

3) John Rycyna (NRC) to Robert Poche (UniStar Nuclear Energy), "RAI No. 65 SEB2 1971.doc (PUBLIC)" email dated February 18, 2009

4) John Rycyna (NRC) to Robert Poche (UniStar Nuclear Energy), "RAI No. 112 SEB 2574.doc (PUBLIC)" email dated April 30, 2009

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5) UniStar Nuclear Energy Letter UN#09-257, from Greg Gibson (UniStar Nuclear Energy) to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 58, Seismic Design Parameters, RAI No. 63, Seismic Subsystem Analysis, RAI No. 65, Seismic System Analysis, and RAI 112, Seismic Design Parameters, dated May 29, 2009

The purpose of this letter is to respond to the requests for additional information (RAIs) identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated February 17, 2009 (Reference 1), February 18, 2009 (References 2 and 3), and April 30, 2009 (Reference 4). These RAIs address Seismic Design and Analysis, as discussed in Section 3.7 of the Final Safety Analysis Report (FSAR), as submitted in Part 2 of the CCNPP Unit 3 Combined License Application (COLA), Revision 4.

References 1, 2, 3, and 4 requested UniStar Nuclear Energy to respond to the RAIs within 30 days. Reference 5 provided a schedule for the expected response dates.

Enclosure 1 provides the summary for our responses to these RAIs.

Enclosure 2 provides our responses to RAI No. 58, Question 03.07.01-9, RAI No. 65, Questions 03.07.02-2, 03.07.02-3, 03.07.02-5, 03.07.02-7, 03.07.02-8, 03.07.02-11, 03.07.02-13, 03.07.02-17, 03.07.02-19, 03.07.02-20, 03.07.02-23, and 03.07.02-26. The responses to RAI No. 65 Questions 03.07.02-17 and 03.07.02-19 will require a follow-up response as indicated in Enclosure 1 and 2. The responses to RAI No. 65, Question 03.07.02-13, 03.07.02-23, and 03.07.02-26 include revised COLA content. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

Enclosure 3 provides oversized tables in support of the response to RAI 65, Question 03.07.02-8.

Our responses do not include any new regulatory commitments.

If there are any questions regarding this transmittal, please contact me at (410) 470-4205, or Mr. Michael J. Yox at (410) 495-2436.

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

Executed on June 12, 2009

Greg Gibson

Enclosures:

1) Response Summary for Requests for Additional Information, RAI No. 58, Seismic Design Parameters, RAI No. 63, Seismic Subsystem Analysis, RAI No. 65, Seismic System Analysis, and RAI No. 112, Seismic Design Parameters, Calvert Cliffs Nuclear Power Plant Unit 3

2) Response to NRC Request for Additional Information, RAI No. 58, Seismic Design Parameters, Question 03.07.01-9, and RAI No. 65, Seismic System Analysis, Questions 03.07.02-2, 03.07.02-3, 03.07.02-5, 03.07.02-7, 03.07.02-8, 03.07.02-11, 03.07.02-13, 03.07.02-17, 03.07.02-19, 03.07.02-20, 03.07.02-23, and 03.07.02-26, Calvert Cliffs Nuclear Power Plant, Unit 3

3) Enlarged tables associated with RAI No. 58 - Seismic Design Parameters, Question 03.07.02-8; Table 1, Summary of Moment Capacities and Demands for Structural Elements of Intake Structure (IS) and Pumphouse (PH); and Table 2, Summary of Shear Capacities and Demands for Structural Elements of Intake Structure (IS) and Pumphouse (PH), Calvert Cliffs Nuclear Power Plant Unit 3

cc: John Rycyna, NRC Project Manager, U.S. EPR COL Application
Laura Quinn, NRC Environmental Project Manager, U.S. EPR COL Application
Getachew Tesfaye, NRC Project Manager, U.S. EPR DC Application (w/o enclosure)
Loren Plisco, Deputy Regional Administrator, NRC Region II (w/o enclosure)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2
U.S. NRC Region I Office

GTG/TD/jmm

Enclosure 1

Response Summary for Requests for Additional Information, RAI No. 58, Seismic Design Parameters, RAI No. 63, Seismic Subsystem Analysis, RAI No. 65, Seismic System Analysis, and RAI No. 112, Seismic Design Parameters Calvert Cliffs Nuclear Power Plant Unit 3

Response Summary for Requests for Additional Information

RAI Set 58 Question	Description of RAI Item	Response Date
03.07.01-1	Justify assumptions of rigid basemat in SSI analysis of Nuclear Island including lower bound soil properties (where shear wave velocity is less than 1000 fps)	September 15, 2009
	Identify impact on the SSI analysis results and on the design of the foundation mat and supported superstructure.	September 15, 2009
03.07.01-2	Provide a figure in the FSAR to depict SSI model of Nuclear Island including the model of subgrade.	July 15, 2009
	State whether or not embedment effects were considered in this analysis and, if not, what is the justification for not including them and what impact could this have on the analysis results.	September 15, 2009
	Describe the properties of the structural backfill and how the fill was modeled in the SSI analysis.	July 15, 2009
	As the groundwater table is close to the bottom of the base mat, how are groundwater effects treated in the SSI confirmatory analysis.	July 15, 2009
	Identify computer codes to perform SSI analysis of NI; provide description of codes, extent of application and basis for validation.	July 15, 2009
	Provide similar information on computer codes used in the generation of FIRS for each Category I structure.	July 15, 2009
	Provide similar information on computer codes used in seismic analysis in Section 3.7.1,3.7.2, and 3.7.3.	July 15, 2009
03.07.01-3	For EPGB and ESWB, provide methodology to calculate FIRS at grade elevation computed from the GMRS which were determined at an and applicable elevation 41 ft below grade.	August 29, 2009
	Describe computer codes, soil column model, and the basis for the shear, wave velocity of the structural backfill that supports both the EPGB and ESWB and the impact of this backfill on the development of the FIRS.	December 29, 2009

Response Summary for Requests for Additional Information

RAI Set 58 Question	Description of RAI Item	Response Date
	Provide in the FSAR the spectra at the foundation level of each structure meeting Appendix S requirements.	December 29, 2009
	Provide in the FSAR a comparison of the FIRS at the foundation level of each structure meeting the requirements of Appendix S to the CSDRS provided in the U.S. EPR FSAR.	December 29, 2009
	Provide the basis for not performing confirmatory analysis for the EPGB and ESWB similar to that for NI.	July 29, 2009
03.07.01-4	In FSAR Section 3.7.1.1.1, on page 3.0-32, it discusses the design response spectrum used to analyze the Ultimate Heat Sink (UHS) Makeup Water Intake Structure. The spectral comparison between the European Utility Requirements (EUR) soft soil spectrum scaled to 0.15 g, the RG 1.60 spectrum scaled to 0.1 g, and the ground motion response spectra (GMRS) shown in Fig. 3.7-38 indicates that the RG 1.60 spectrum and GMRS exceed the EUR spectrum at frequencies below 0.7 and 0.4, respectively. What is the corresponding comparison of displacements and velocities for these spectrum motions, and if the EUR displacements are exceeded, how will this be addressed in the design of piping and other appurtenances connected to these buildings including the design of buried utilities?	July 15, 2009
03.07.01-5	For Ultimate Heat Sink Electrical Building, provide and include in the RAI response FSAR the horizontal and vertical spectra depicting design spectra and applicable envelope.	August 29, 2009
	Provide in the FSAR a reconciliation of the design response spectrum with the horizontal foundation input response spectra (FIRS) for this structure which meets the minimum requirements of 10 CFR Part 50, Appendix S.	December 29, 2009
	Include a description of how the FIRS are developed including the soil model, soil properties, backfill properties, computer programs and analysis assumptions.	December 29, 2009
03.07.01-6	Provide in the FSAR how the design response spectrum and assumed soil properties used in the analysis of the UHS MWIS will be reconciled with the FIRS that meets the requirements of Appendix S and the final soil properties determined from the site final geotechnical studies.	September 14, 2009
	Include in the FSAR a comparison of the FIRS with the design response spectra used in the analysis.	December 29, 2009

Response Summary for Requests for Additional Information

RAI Set 58 Question	Description of RAI Item	Response Date
	Include a description of how the FIRS are developed including the soil model, soil properties, computer programs, and analysis assumptions.	December 29, 2009
03.07.01-7	Provide in the FSAR a discussion of the site-specific spectra that were considered for buried utilities.	December 29, 2009
	Provide justification for the use of the EUR soft soil spectrum including possible displacement and velocity differences that may exist with the use of this spectrum as opposed to using a site specific spectrum.	December 29, 2009
	Provide a comparison of the EUR soft soil spectrum with appropriate site specific spectra that are applicable to buried utilities.	December 29, 2009
03.07.01-8	See UniStar Nuclear Energy letter UN#09-228, dated May 1, 2009	Response submitted
03.07.01-9	This Letter – See Enclosure 2.	Response submitted
03.07.01-10	State explicitly or by reference design ground motion time histories for RAI partial Nuclear Island, EPGB and ESWB structures.	September 15, 2009
	What are the site specific design ground motions and their bases that apply to these structures? Provide this information in Section 3.7.1.1.2 of the FSAR.	December 29, 2009

Response Summary for Requests for Additional Information

RAI Set 63 Question	Description of RAI Item	Response Date
03.07.03-1	For the analysis of buried utilities, provide the following information: • Describe any computer codes used for the analysis and their application to the analysis and design of buried utilities. • Provide the soil properties used in the analysis and explain how differences in soil properties were accommodated in the analysis. • Provide the design codes and acceptance criteria for each category of buried utilities. • Describe the missile protection provided for safety-related buried utilities. • Describe how ground water effects were considered in the analysis. • For utility runs that are both above and below ground, describe how above ground inertial effects were combined with below ground seismic wave effects. • Describe how the wave velocities were determined for calculating the maximum axial strain. • Provide the basis for determining the maximum friction force per unit length of pipe.	July 15, 2009
	For the analysis of buried utilities, provide the following information: Describe how the building anchor point displacements were determined and how these were combined with seismic wave effects and soil loads	December 15, 2009

Response Summary for Requests for Additional Information

RAI Set 65 **Question**	**Description of RAI Item**	**Response Date**
03.07.02-1	See UniStar Nuclear Energy letter UN#09-228, dated May 1, 2009	Response submitted
03.07.02-2	This Letter – See Enclosure 2.	Response submitted
03.07.02-3	This Letter – See Enclosure 2.	Response submitted
03.07.02-4	Provide results of SSI analysis for Ultimate Heat Sink Electrical Building that meet the acceptance criteria 4.A.vii of SRP 3.7.1 and acceptance criteria 4 of SRP 3.7.2 using subgrade model of final soil and backfill properties or justify alternative.	December 29, 2009
	Include SSSI effects from UHS MWIS.	December 29, 2009
	Reconcile with the results of assumed seismic response and ISRS.	December 29, 2009
03.07.02-5	This Letter – See Enclosure 2.	Response submitted
03.07.02-6	Describe how the SSI analysis for Ultimate Heat Sink Makeup Water Intake Structure (UHS MWIS) performed meets the acceptance criteria and 4.A.vii of SRP 3.7.1 or justify alternative.	December 29, 2009
	Provide a figure depicting the soil-structure model used for the seismic analysis.	December 29, 2009
	Provide the basis for the assumed soil properties and profile used to calculate the frequency independent impedance functions.	August 15, 2009
	Provide the method and formulas used to calculate the values of the soil springs under the foundation as well as the lateral soil springs that represent the embedment effects.	August 15, 2009

Response Summary for Requests for Additional Information

RAI Set 65 Question	Description of RAI Item	Response Date
	State whether the soil properties used in the analysis are strain dependent or simply the low strain values. If these are low strain values, justify their use and quantify the impact of not using strain dependent properties on the results of the analysis. If the soil properties are strain dependent, describe how the final soil properties are determined in the analysis.	August 15, 2009
	For large values of Poisson's ratio, the dynamic stiffness and damping are frequency dependent. Provide justification for assuming that the impedance functions of the supporting foundation are frequency independent.	August 15, 2009
	Confirm that the control motion is applied at the base of the soil structure analysis model.	August 15, 2009
	Provide a reconciliation of the final soil properties and the foundation input response spectra (FIRS) that are based on these properties with the seismic analysis results described in the FSAR.	December 29, 2009
03.07.02-7	This Letter – See Enclosure 2.	Response submitted
03.07.02-8	This Letter – See Enclosure 2.	Response submitted
03.07.02-9	See UniStar Nuclear Energy letter UN#09-126, dated March 19, 2009	Response submitted
03.07.02-10	See UniStar Nuclear Energy letter UN#09-228, dated May 1, 2009	Response submitted
03.07.02-11	This Letter – See Enclosure 2.	Response submitted
03.07.02-12	Provide results of a structure-to-structure interaction analysis between UHS MWIS and EB.	December 29, 2009
03.07.02-13	This Letter – See Enclosure 2.	Response submitted
03.07.02-14	See UniStar Nuclear Energy letter UN#09-228, dated May 1, 2009	Response submitted

Response Summary for Requests for Additional Information

RAI Set 65 Question	Description of RAI Item	Response Date
03.07.02-15	In FSAR Section 3.7.2.6 on page 3.0-40, it states that for the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS), three statistically independent time histories are applied for each of the six soil cases to determine accelerations at select locations. Describe how the accelerations obtained from this dynamic analysis are applied to the static model to obtain forces and moments for structural design and provide examples of how the three components of earthquake motion are combined and compare the results to those of the 100-40-40 rule presented in RG 1.92, Revision 2. The use of an equivalent static approach to determine forces and moments in the structure may not be conservative as dynamically computed forces and moments will retain the appropriate sign from the analysis and the static approach will not. How will this be addressed in the development of loads used in the design of the structure?	July 15, 2009
03.07.02-16	See UniStar Nuclear Energy letter UN#09-126, dated March 19, 2009	Response submitted
03.07.02-17	This Letter – See Enclosure 2. Follow-up document transmittal required.	See RAI response this submittal
03.07.02-18	Clarify the seismic classification of fire protection tank and building.	July 29, 2009
	Reconcile the U.S. EPR seismic analysis for NAB with the site-specific soil properties and foundation input response spectra (FIRS)	September 15, 2009
	Demonstrate in the FSAR that the displacement of this structure relative to the nuclear island common basemat structure is enveloped by the results of the U.S. EPR analysis.	September 15, 2009
03.07.02-19	This Letter – See Enclosure 2. Follow-up document transmittal required	See RAI response this submittal
03.07.02-20	This Letter – See Enclosure 2.	Response submitted
03.07.02-21	See UniStar Nuclear Energy letter UN#09-228, dated May 1, 2009	Response submitted
03.07.02-22	See UniStar Nuclear Energy letter UN#09-126, dated March 19, 2009	Response submitted

Response Summary for Requests for Additional Information

RAI Set 65 Question	Description of RAI Item	Response Date
03.07.02-23	This Letter – See Enclosure 2.	Response submitted
03.07.02-24	Per COLA item 3.7-1, address that the seismic response of the nuclear island common base mat structures, seismic Category II structures, the Nuclear Auxiliary Building and the Radioactive Waste Processing Building is within the parameters of Section 3.7 of U.S. EPR FSAR.	September 15, 2009
	Provide a summary for each structure, either directly or by reference, September 15, which describes how the COL item is met.	September 15, 2009
03.07.02-25	See UniStar Nuclear Energy letter UN#09-228, dated May 1, 2009	Response submitted
03.07.02-26	This Letter – See Enclosure 2.	Response submitted

Response Summary for Requests for Additional Information

RAI Set 112 Question	Description of RAI Item	Response Date
03.07.01-11	Provide a definition of site SSE and explain how it meets regulation requirements.	September 15, 2009
	Consistent with the site SSE, provide the FIRS in the free field at the foundation level of each structure meeting the requirements of Appendix S, and describe how each is determined.	September 15, 2009 (NI) December 15, 2009 (EPGB, ESWB)
	For the U.S. EPR Certified Design structures, provide a comparison of the results of the site seismic analyses using the FIRS input motion defined at the foundation level of each structure, with the analyses results documented in the U.S. EPR FSAR.	September 15, 2009 (NI) December 29, 2009 (EPGB, ESWB)
	For the EPGB and ESWB, describe how the effect of structure-soil-structure interaction has been accounted for in the analysis of these buildings.	December 29, 2009 (EPGB, ESWB)

Enclosure 2

Response to NRC Request for Additional Information, RAI No. 58, Seismic Design Parameters, Question 03.07.01-9, and RAI No. 65, Seismic System Analysis, Questions 03.07.02-2, 03.07.02-3, 03.07.02-5, 03.07.02-7, 03.07.02-8, 03.07.02-11, 03.07.02-13, 03.07.02-17, 03.07.02-19, 03.07.02-20, 03.07.02-23, and 03.07.02-26, Calvert Cliffs Nuclear Power Plant Unit 3

RAI No. 58

Question 03.07.01-9

FSAR Section 3.7.1.1.1, page 3.0-32 characterizes the geotechnical data as preliminary. In general, noted throughout FSAR Section 3.7 there are issues that are to be resolved in the final detailed design. It is not clear how the site-specific structures will meet the requirements of GDC 2. Provide a table that lists the items to be resolved in the final detailed design, how the items will be closed, and how these are to be incorporated into the final version of the FSAR.

Response

The items associated with the analysis of site-specific structures, namely, Ultimate Heat Sink Makeup Water Intake Structure (UHS MWIS) and Ultimate Heat Sink Electrical Building (UHS EB), noted as preliminary in Sections 3.7.1 and 3.7.2, are tabulated below. The table also notes how a specific item would be closed including incorporation of related analysis information into the FSAR. Additionally, the table identifies general open items in Section 3.7.

Item No.	**Issues (Site-Specific Structures)**	**Method of Closure**	**Incorporation into FSAR**
1	Geotechnical Data for UHS MWIS and UHS EB • Soil profile information including backfill • Static soil properties - density, Poisson's ratio, low strain shear wave velocity • Dynamic soil properties – strain compatible shear wave velocities, P wave velocities and damping • Ground water table	The geotechnical information will be developed after subsurface investigations and Resonant Column Torsional Shear testing are complete.	FSAR Section 2.5.2, 2.5.4, 3.7.1 and 3.7.2 to be updated accordingly.
2	FIRS for UHS MWIS	FIRS for UHS MWIS will be developed consistent with the Ground Motion Response Spectrum in accordance with NRC regulations.	FIRS to be included in Section 3.7.1.
3	FIRS for UHS EB	FIRS for UHS EB will be developed consistent with GMRS and in accordance with NRC regulations.	FIRS to be included in Section 3.7.1.

Item No.	Issues (Site-Specific Structures)	Method of Closure	Incorporation into FSAR
4	Soil-Structure Interaction Analysis of UHS EB	Soil-Structure Interaction Analysis using SASSI will be performed for UHS EB after appropriate inclusion of SSSI effects from UHS MWIS.	Results of SASSI analysis to be included in Section 3.7.2 following the guidelines of RG 1.206.
5	Soil-Structure Interaction Analysis using SASSI for UHS MWIS	• Existing SSI analysis using modal superposition time history analysis method will be reconciled using SASSI. • Equivalent static accelerations and In-structure response spectra reported in FSAR will be reconciled accordingly with results from SASSI. • The SASSI analysis will also reconcile the use of uncracked section properties. • The negligible impact of convective forces will be reconciled for structural and component design.	Results of existing SSI analysis to be reconciled in Section 3.7.2.

Item No.	Issues (Site-Specific Structures)	Method of Closure	Incorporation into FSAR
6	Interaction of Non-Seismic Category I Structures with Seismic Category I SSCs • Circulating Water Intake Structure • Retaining Wall (forebay) and existing non-seismic bulkhead • Aboveground Seismic Category II and Seismic Category II-SSE Fire Protection SSC • Buried Seismic Category II-SSE Fire Protection SSC	• Closure of ITAAC (Table 2.4-19 of COLA Part 10) associated with Circulating Water Intake Structure will ensure that this structure does not adversely interact with Category I SSC. • The retaining wall (forebay), along with UHS MWIS and EB, has been relocated to the south of Unit 1 and 2 forebay. Closure of ITAAC (Table 2.4-7) of COLA Part 10 associated with the retaining wall will ensure that this structure does not adversely interact with Category I SSC. • The existing non-seismic bulkhead, due to its location, will not adversely interact with UHS MWIS and EB. • Closure of ITAAC (Tables 2.4-10, 2.4-26 and 2.4-27 of COLA Part 10) associated with buried and above ground fire protection SSCs will confirm that these commodities will remain functional during and following an SSE in accordance with RG 1.189.	• Section 3.7.2.8 to be updated accordingly for circulating water intake structures. • Sections 3.7.2.8 to be updated accordingly for the retaining wall. • Section 3.7.2.8 to be updated accordingly for existing non-seismic bulkhead. • Section 3.7.2.8 to be updated accordingly for buried and above ground fire protection SSCs.

COLA Impact

None

RAI No. 65

Question 03.07.02-2

In FSAR Section 3.7.2.1.4 (Equivalent Static Load Method of Analysis) on page 3.0-35, it states that the equivalent static load method is used for the UHS EB by applying 0.5 g acceleration in all directions. Assuming the zero period acceleration (ZPA) of the design input ground motion is .35 g, provide the justification for the amplification of ground acceleration used for this structure, i.e. .5/.35, or 1.43. In addition, an assumption is made that the walls and slabs are stiff. This is used as the basis for assuming there is no additional amplification of the seismic response of the structure due to local flexibility of the structural elements. While it may be true the in-plane stiffness of the walls and slabs exceed 33 Hz, it may not be true that this is the case for their out-of-plane response. Provide the results of an analysis that demonstrates that the out-of-plane response for walls and slabs exceeds 33 Hz. Include in this analysis technical consideration of whether the walls and slabs are cracked or uncracked under the applied design loads.

Response

Justification for Use of 0.5 g Acceleration for the Equivalent Static Analysis

As described in response to RAI 65 Question 03.07.02-14[1], the Ultimate Heat Sink (UHS) Electrical Building (EB) is a rigid structure that is essentially fully embedded. The seismic response of the UHS EB is governed by compatibility with the surrounding soil medium, and as such, soil governed mode shapes control the response of the structure. For design of the UHS EB, the associated frequencies of the soil driven mode shapes were conservatively considered to lie on the peak of the design response spectrum, which is taken as the envelope of the EUR Soft Spectrum scaled down to a ZPA of 0.15 g and the In-Structure Response Spectrum (ISRS) of the adjacent UHS Makeup Water Intake Structure (MWIS) at the operating deck level with a ZPA of 0.35 g. In the absence of more accurate information concerning the structure-soil-structure interaction (SSSI) between the UHS MWIS and UHS EB, the UHS MWIS operating deck ISRS, which has a ZPA of 0.35 g, is used without any reduction to conservatively account for the resulting SSSI effects. Considering at least 20% damping for the soil driven modes, the maximum structural acceleration response, based on the amplification factors using Reference 1 of RG 1.60, will not exceed the considered acceleration of 0.50 g applied in all directions using the equivalent static method. The structural response will be confirmed using SASSI analysis.

Consideration of Out-of-Plane Flexibility and Cracking of the Structural Elements

The plant northwest exterior wall of the UHS EB, which is shown in FSAR Figures 3E.4-5 and 3.8-1, is determined to be the most flexible of the UHS EB walls and slabs for out-of-plane vibrations. The fundamental frequency of the wall is calculated using Rayleigh's Method for

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[1]UniStar Nuclear Energy Letter UN#09-228, from Greg Gibson (UniStar Nuclear Energy) to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 58, Seismic Design Parameters, RAI No. 63, Seismic Subsystem Analysis, RAI No. 65, Seismic System Analysis, dated May 1, 2009

plates, as described by Blevins[2]. Using a length of 70 ft, height of 14 ft and thickness of 2 ft, the out-of-plane fundamental frequency of the wall is calculated using two sets of boundary conditions, one with sides fixed and one with sides simply-supported, representing an upper and lower bound, respectively. Using fixed boundary conditions with an uncracked section, the fundamental frequency is approximately 120 Hz. For simply-supported boundary conditions with an uncracked section, the fundamental frequency is approximately 54 Hz. If the section is assumed to be cracked for the simply-supported case, then the fundamental frequency is reduced to approximately 38 Hz, which still exceeds the ZPA cutoff frequency of 33 Hz. Therefore, both boundary condition assumptions, whether in the cracked or uncracked condition, result in a wall fundamental frequency that exceeds the ZPA cutoff frequency of 33 Hz. Since the plant northwest exterior wall is the most flexible, other structural elements will exhibit higher fundamental frequencies for out-of-plane vibration. Therefore, the structure behaves in a rigid manner for local out-of-plane response, and there is no additional amplification owing to out-of-plane flexibility.

COLA Impact

None

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[2] R. D. Blevins, "Formulas for Natural Frequency and Mode Shapes," Krieger Publishing Company, 1984

RAI No. 65

Question 03.07.02-3

As shown in the applicant's FSAR, no specific dynamic analysis has been performed for the Ultimate Heat Sink Electrical Building. How are the building displacements calculated which are needed as inputs for the analysis of buried conduit, duct banks, and piping that interface with this structure?

Response

At this stage of the design, no specific dynamic analysis has been performed for the Ultimate Heat Sink (UHS) Electrical Building (EB) to obtain building displacements needed as inputs for analyzing buried commodities, such as buried conduit, duct banks, and piping. However, as described in FSAR Section 3.7.2.4, during detailed design a geotechnical site investigation and study will be conducted to confirm geotechnical data and seismic parameters. These confirmed design inputs will be used in SSSI analyses to quantify building seismic relative displacements between the UHS Makeup Water Intake Structure (MWIS) and UHS EB. Displacements are then computed at locations where the buried commodities interface with the UHS EB.

COLA Impact

None

RAI No. 65

Question 03.07.02-5

In FSAR Section 3.7.2.3.2 (Seismic Category I Structures - Not on Nuclear Island Common Base Mat) on page 3.0-36, it describes the finite element model used in the analysis of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS).

- SRP 3.7.2, SRP Acceptance Criteria 3.C.ii. states the element mesh size should be selected on the basis that further refinement has only a negligible effect on the solution results. Describe any sensitivity studies that were implemented in determining the mesh size for the UHS MWIS, and if no sensitivity study was performed provide justification for not doing so.

- SRP 3.7.2, SRP Acceptance Criteria 3.D. states that in addition to the structural mass, a floor load of 244.64 kg/m2 (50 pounds/ft^2) should be included to represent miscellaneous dead weights and a mass equivalent to 25 percent of the floor design live load and 75 percent of the roof design snow load should be included in the model. Describe how this acceptance criterion has been addressed in the model of the UHS MWIS, and if no additional mass was added provide the justification for not doing so.

Response

Describe any sensitivity studies that were implemented in determining the mesh size for the UHS MWIS, and if no sensitivity study was performed provide justification for not doing so.

The average element size (GT STRUDL element type SBHQ6) in the finite element model of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS), described in FSAR Section 3.7.2.3.2 and shown in FSAR Figure 3.7-37, is approximately 3 ft with a maximum element size of 4.4 ft. The foundation base mat and the embedded exterior side wall of UHS MWIS are identified as the critical design elements as discussed in FSAR Appendix 3E.4. The thickness of these structural elements is 4 ft and the maximum finite element size used is 3.5 ft. It is further noted that these critical structural elements do not contain any openings and are not subjected to any concentrated loads, so that stress concentration effects in the structural elements are negligible and their influence on the response is insignificant. As the purpose of the static analysis is to generate element-by-element out-of-plane moment and shear responses and in-plane shear and membrane loads for concrete reinforcement design, a maximum element size of less than "*1t*", where *t* is the element thickness, represents sufficient refinement. This element size is well within the normally accepted limits over which stress resultants in concrete slabs and panels are averaged.

For dynamic response, the mesh refinement of the finite-element model needs to be sufficient to enable determination of the local and global dynamic modes and frequencies. The out-of-plane fundamental frequencies of the critical structural elements exceed the zero period acceleration (ZPA) frequency of 33 Hz. A minimum of five elements in each direction, between the supporting edges, are used to model the structural panels. This mesh density is sufficient to capture the fundamental frequencies and mode shapes of critical structural elements. It is further noted that this mesh density is refined to model any curvature reversals that might occur between panel boundaries. For each of the six soil cases described in FSAR Section 3.7.2.4, at least 99.90% of modal mass was captured in each direction. There is no effect due to missing modal mass on slabs and panels structural responses and in-structure response spectrum (ISRS).

Therefore, for the reasons mentioned above, no further mesh sensitivity was performed.

Describe how SRP 3.7.2 Acceptance Criterion 3.D has been addressed in the model of the UHS MWIS, and if no additional mass was added, provide the justification for not doing so.

As stated in FSAR Section 3E.4.1, 25% of the design live load and 75% of the design snow load, as described in Acceptance Criteria 3.D of SRP 3.7.2, are included in the time history analysis and the structural design.

The seismic analysis considered 3 psf additional floor load to account for miscellaneous piping, cable trays and HVAC ducts. This load was based on the planned routing and layout of the miscellaneous commodities. The subsequent equivalent static analysis and structural design considered 50 psf additional floor load in accordance with the recommended values cited in Acceptance Criteria 3.D of SRP 3.7.2.

The additional floor dead load of 47 psf included in the equivalent static analysis is less than 1% of the total mass. The corresponding decrease in global structural frequencies is, therefore, negligible and has an insignificant effect on the corresponding spectral accelerations used in the equivalent static analysis.

Although application of the recommended 50 psf floor load would reduce local out-of-plane frequencies of the operating deck by approximately 6%, the reduced frequencies still remain above the ZPA frequency. Therefore, local flexibility of these elements will not result in additional amplification of the seismic responses.

The miscellaneous floor load of 50 psf will be included in the System for Analysis of Soils Structure Interaction (SASSI) analysis of UHS MWIS. As noted in the response to RAI 58 Question 03.07.01-9 (this enclosure), the results of the existing time history analysis will be reconciled by SASSI analysis.

COLA Impact

None

RAI No. 65

Question 03.07.02-7

In FSAR Section 3.7.1.1 (pg 3.0-29), it indicates that the Category I makeup water intake structure (MWIS) is founded below sea level. The description of the soil-structure-interaction (SSI) analysis for this structure does not describe how the ground water effects were included in the analysis. Describe how the SSI calculations included these effects, and if they did not, provide justification for not doing so and address the impact.

Response

The effect of the ground water table is included in the UHS Makeup Water Intake Structure (MWIS) time history analysis by considering the saturated soil properties. The frequency independent lumped foundation impedances are determined, as described in FSAR Section 3.7.2.4, using a Poisson's ratio of 0.47, which represents the saturated soil condition for the founding media under UHS MWIS.

As noted in FSAR Section 3.7.2.4, the analysis is based on preliminary soil data, and geotechnical investigation will be conducted during detailed engineering design to evaluate the effects of ground water variability on the compression wave velocity. Additionally, as addressed in response to RAI 58 Question 03.07.01-9 (this enclosure), a soil-structure-interaction (SSI) analysis using SASSI will be performed to reconcile the frequency independent impedance functions.

COLA Impact

None

RAI No. 65

Question 03.07.02-8

FSAR Section 3.7.2.3.2 states that the Ultimate Heat Sink Makeup Water Intake Structure is analyzed in GTSTRUDL. It further states that the walls "are not anticipated" to crack. Provide the basis for this statement including numerical results for typical concrete sections using the applicable wall design loads.

Response

The Ultimate Heat Sink (UHS) Makeup Water Intake Structure is described below using a simplified sketch (Figure 1). The below-grade portion, referred to as the Intake Structure, has two side walls (North-South direction), one back wall (East-West direction) and an operating deck (interior walls not shown for clarity). The above-grade portion, referred to as the Pumphouse, located on top of the Intake Structure, has two side walls (North-South direction), one back wall and one front wall (East-West direction) and roof slab. As stated in the FSAR Section 3.7.2.3.2, the time history analysis is based on the un-cracked concrete section properties. FSAR Section 3.7.2.3.2 also notes that for the two North-South exterior walls (referred to as side wall of intake structure in this RAI response), cracking may occur during the SSE condition. However, overall impact to the global seismic response is determined to be insignificant. The results presented in this RAI response are based on the enveloped (seismic and non-seismic) loading conditions. It is also noted that the results due to seismic loading demands are based on conservative equivalent static method of analysis, where seismic accelerations obtained from the time history analysis are based on the peak values applied in-phase. It is demonstrated that different structural elements remain un-cracked for out-of-plane bending moment demands and shear demands. As noted in FSAR Section 3.7.2.3.2 and as indicated in the response to RAI 58 Question 03.07.01-9 (this enclosure), the use of un-cracked section properties in the seismic analysis will be verified by soil-structure interaction analysis using SASSI. Subsequent paragraphs of this response elaborate on the methodology to check the un-cracked condition of various structural elements and present numerical results.

For out-of-plane bending, the un-cracked condition of each structural element is verified by comparing the maximum moment demand (M_a) and the cracking moment (M_{cr}) per ACI-349-01 Section 9.5.2.3. The moment demand is calculated by adding absolute values of the out-of-plane bending (M_{xx} or M_{yy}) and normal twisting (M_{xy}) moments for the governing (seismic and non-seismic) load combination. The notation for the moment resultants is defined in FSAR Figure 3E.4-7. Numerical results of the verification are presented in Table 1 (Enclosure 3). For the north-south exterior wall only, results are presented for seismic and non-seismic loading combinations. For other structural elements, results are presented for the worst case loading condition.

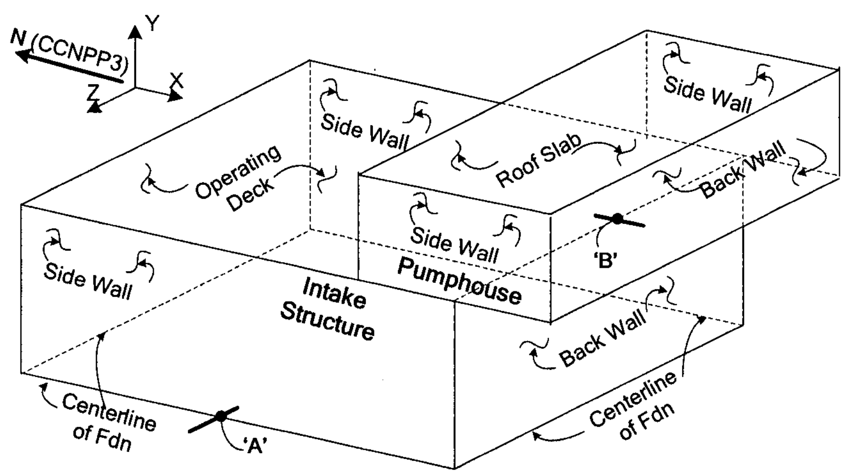
For in-plane and out-of-plane shear, each structural element is verified by comparing the factored shear force (V_u) and the nominal shear strength (V_c) provided by concrete. Numerical results of the verification are presented in Table 2 (Enclosure 3). For the North-South exterior wall only, results are presented for seismic and non-seismic loading combinations. For other structural elements, results are presented for the worst case loading condition.

Table 1 and Table 2 (Enclosure 3) show that the values of (Mcr/M_a) and (V_c/V_u), respectively, for each structural elements are greater than 1.0. Therefore, using un-cracked properties for the walls, deck and roof slab in accordance with ACI-349-01 Section 9.5.2.3 and ASCE 43-05 Table 3-1 as input in the GTSTRUDL finite element model for seismic analysis is appropriate.

COLA Impact

None

Figure 1, Isometric View of UHS Makeup Water Intake Structure



RAI No. 65

Question 03.07.02-11

In FSAR Section 3.7.2.4 on page 3.0-37, it states that the convective frequencies associated with sloshing effects occur in the range where the scaled down European Utility Requirements (EUR) spectra do not exceed either the CCNPP Unit 3 spectra (zero period acceleration (ZPA) of 0.067 g) or Regulatory Guide 1.60 spectra scaled to a ZPA of 0.10 g. It goes on to say that due to the lower acceleration levels at the convective frequencies and the lower convective water mass, the convective forces are anticipated to be minimal with respect to the impulsive forces. If the foundation input response spectra (FIRS) for this structure are the scaled down EUR spectra, explain why this is an appropriate response spectra for this site when the low frequency input is less than that of the ground motion response spectra (GMRS) which has a ZPA of .067 g. What is the basis for the calculation of the convective water mass? Why was this mass not included in the analysis of the UHS MWIS? How will the difference in input response spectra be resolved in determining the proper convective design loads for the structure?

Response

Explain why the scaled down EUR spectra are appropriate for this site when the low frequency input is less than that of the ground motion response spectra (GMRS) which has a ZPA of .067 g.

As stated in FSAR Section 3.7.1.1.1 (pages 3-29 and 3-302) and as shown in FSAR Figure 3.7-38, the scaled down EUR Soft spectrum is not enveloped by the RG 1.60 spectrum anchored at 0.1 g or the GMRS with ZPA of 0.067 g for frequencies below 0.63 Hz. Since 0.63Hz is much smaller than the lowest natural frequency (2.60 Hz per FSAR Tables 3.7-7 through 3.7-11) of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS), deficiencies in the low frequency range have negligible impact on the soil-structure-interaction (SSI) analyses of the intake structure. However, convective force magnitudes and oscillation wave height due to sloshing are evaluated for the interior water chambers and the exterior forebay to assess impact on the structural design.

As indicated in FSAR Section 3.7.2.4 (page 3-35), the convective forces associated with the sloshing frequencies (0.30 Hz and 0.51 Hz) of contained water inside the intake structure chambers are small due to low acceleration values (0.14g max, conservatively at 0.5% damping, corresponding to 0.51 Hz) obtained from the enveloping RG 1.60 spectrum anchored at 0.1 g. The convective forces are however, conservatively captured in the static analysis model of the structure by assigning 0.5g acceleration to the impulsive water mass (which conservatively envelops the convective forces since the peak of the scaled down EUR Soft spectrum is at 0.45 g). This approach ensures that deficiencies in the low frequency range of the scaled down EUR Soft spectrum are captured as far as convective forces are concerned.

Using the acceleration values obtained from the RG 1.60 spectrum anchored at 0.1 g and Equation (7-1) of Section 7.1 of ACI 350.3-06, the maximum calculated oscillation wave height corresponding to earthquake motion in either direction is approximately 1.2 ft. The available freeboard sufficiently exceeds the calculated oscillation wave height to preclude any hydrodynamic load onto the underside of the operating deck slab.

What is the basis for the calculation of the convective water mass?

The convective masses associated with the sloshing effects of the contained water inside the intake structure chambers are calculated in accordance with Equation (9-2) of Section 9.2.1 of ACI 350.3-06.

The convective mass effects of the forebay water were calculated using the methodology described in Section 2-19 of the U.S. Army Corps of Engineer Manual EM 1110-2-6051. The frequency of the convective mass due to sloshing of the forebay water was less than 0.15 Hz. Due to the negligible magnitude of the associated spectral acceleration, this mass was not considered in the further static or dynamic analyses of the UHS MWIS.

Why was this (the convective water mass) not included in the analysis of the UHS MWIS?

As stated in the preceding paragraph, the convective masses are of minor magnitude compared to the impulsive masses (less than one-fourth), and are insignificant compared to the total weight of the structure. Furthermore, the low frequency values associated with the convective modes (0.51 Hz max) signify that the convective mode shapes are uncoupled with any soil-governed or structural mode shapes. With respect to the overall seismic response of the intake structure, the response due to the convective modes will be small and localized in the upper reaches of the walls. As such, inclusion or exclusion of the convective masses has negligible impact on the overall response of the structure.

How will the difference in input response spectra be resolved in determining the proper convective design loads for the structure?

Convective forces and oscillation wave heights, which are the response quantities of interest associated with the convective mode shapes, are calculated separately. The effect of the convective forces on the design is captured using 0.5 g acceleration applied to the impulsive water mass. The oscillation wave height is calculated using the acceleration values from the RG 1.60 spectrum anchored at 0.1 g to verify that the available freeboard exceeds the maximum slosh wave height.

COLA Impact

None

RAI No. 65

Question 03.07.02-13

In FSAR Section 3.7.2.6 (Three Components of Earthquake Motion) on page 3.0-40, it states for the Ultimate Heat Sink (UHS) Electrical Building that due to building symmetry cross-coupling is determined to be negligible. As no dynamic analysis was performed for this structure, what is the justification for this statement?

Response

The Ultimate Heat Sink (UHS) Electrical Building (EB) is essentially a fully embedded structure. Frequency analysis of the UHS EB structure showed that the building natural frequencies (three translational and one torsional) are greater than the EUR response spectrum zero period acceleration (ZPA) of 33 Hz.

The floor plan of the UHS EB (FSAR Figure 3E.4-5) shows symmetry about its E-W (short) direction, but with the inclusion of interior walls contributing lateral stiffness, the structure exhibits eccentricity about its N-S (long) direction. The last sentence of the second paragraph of FSAR Section 3.7.2.6 will therefore be revised to account for the interior building geometry. Eccentricity between the building center of gravity and center of rigidity along the E-W (short) building direction was found to be small (approximately 2 ft) compared to the building width (33 ft). However, inherent and accidental torsion caused by eccentricities were calculated and their subsequent cross-coupling effects on the shear forces in the walls determined. Results of this analysis showed that the co-directional shear forces in the walls using the 100-40-40 combination method according to RG 1.92 Rev. 2, increase in-plane shear by approximately 3% (hence negligible) compared with the direct shear forces obtained from the corresponding directional component of earthquake motion. For completeness though, cross-coupling effects are accounted for in the equivalent static analysis of the UHS EB structure.

It is further noted that the inclusion of cross-coupling effects does not affect the thickness or reinforcement in the UHS EB shear walls.

COLA Impact

Part 2, FSAR of the CCNPP Unit 3 COLA will be updated in a future COLA revision to incorporate the changes to FSAR Sections 3.7.2.6 that are identified below:

3.7.2.6 Three Components of Earthquake Motion

For the site-specific UHS Makeup Water Intake Structure, three statistically independent time histories are applied, component by component, to the finite element model for each of the six soil cases to determine accelerations at select locations. An equivalent static analysis is then performed via the finite element model to determine forces and moments for structural component design.

Separate manual calculations, using the equivalent static analysis method, are performed to determine the structural response of the site-specific UHS Electrical Building in each of the three directions. ~~Due to the building symmetry, cross-coupling is determined to be negligible~~ Inherent and accidental torsion due to unsymmetrical layout of the UHS Electrical Building were determined to contribute minor cross-coupling effects.

The equivalent static analyses of both the UHS Makeup Water Intake Structure and the UHS Electrical Building use the ASCE 4-98 (ASCE, 1986) "100-40-40" rule to calculate co-directional response.

RAI No. 65

Question 03.07.02-17

The interaction of non-seismic Category I structures with Seismic Category I systems is described in FSAR Section 3.7.2.8. In this section on page 3.0-41, it states that fire protection SSCs are categorized as either Seismic Category II-SSE, meaning the SSC must remain functional during and after a Safe Shutdown Earthquake (SSE), or Seismic Category II, meaning the SSC must remain intact after an SSE without deleterious interaction with a Seismic Category I or Seismic Category II-SSE SSC. In the U.S. EPR FSAR on page 3.7-95, it states that Seismic Category II is designed to the same criteria as Seismic Category I structures. In SRP 3.7.2, SRP Acceptance Criteria 8, which addresses the interaction of non-Category I structures with Category I SSCs, it states that when non-Category I structures are designed to prevent failure under SSE conditions; the margin of safety shall be equivalent to that of the Seismic Category I structure.

- Describe how this margin of safety is achieved for the Seismic Category II-SSE and Seismic Category II portions of the fire protection system. Include in your response the seismic inputs, loading combinations, codes and acceptance criteria. What are the differences in the method of design for these two seismic categories?

- Describe the basis and provide figures in the FSAR of the design response spectra used to analyze above ground seismic Category II and seismic Category II-SSE fire protection SSCs including the fire protection tanks.

- What are the methods of analysis and acceptance criteria for both the buried and above ground portions of the fire protection system that are Seismic Category II-SSE that will ensure that these portions of the system will remain functional following an SSE event?

- What are the modeling and analysis methods used for the fire protection tanks and to what extent do the fire protection tanks meet the acceptance criteria of SRP 3.7.3, SRP Acceptance Criteria 14.A. thru J.? When the tank analysis does not meet the acceptance criteria, provide the technical justification for not doing so.

Response

The U.S. EPR Design Certification process includes similar RAI questions concerning the interaction of non-seismic structures with seismic Category I structures. AREVA has answered some RAI questions concerning these interactions but has received follow-up NRC questions. AREVA received the follow-up questions on May 19, 2009[3] and is scheduled to respond to the RAI within 30 days of receipt. UniStar Nuclear Energy is working closely with AREVA and Bechtel on these questions concerning the interaction of non-seismic structures with seismic

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[3] Email from the Getachew Tesfaye (NRC) to AREVA, U.S. EPR Design Certification Application RAI No. 215 (2560, 2561, 2565, 2588), FSAR Ch. 3, dated May 19, 2009

Category I structures. The UniStar Nuclear Energy response schedule for answering this RAI 65 question will be provided shortly after the AREVA response.

COLA Impact

None

RAI No. 65

Question 03.07.02-19

In FSAR Section 3.7.2.8 on page 3.0-42 it states that the conventional seismic switchgear building, conventional seismic grids systems control building, the conventional seismic circulating water intake structure and the Seismic Category II retaining wall surrounding the CCNPP Unit 3 intake channel could potentially interact with Seismic Category I SSCs. For each of the above structures, describe in the FSAR how the seismic interaction acceptance criteria of SRP 3.7.2, SRP Acceptance Criteria 8 are met, or justify an alternative. If they are intended to meet criterion B, provide the technical basis for the determination that the collapse of the non-Category I structure is acceptable. For criterion C, confirm that the structure will be analyzed and designed to have a margin of safety equivalent to that of a Category I structure and state how this will be accomplished.

Response

The U.S. EPR Design Certification process includes similar RAI questions concerning the interaction of non-seismic structures with seismic Category I structures. AREVA has answered some RAI questions concerning these interactions but has received follow-up NRC questions. AREVA has received the follow-up questions on May 19, 2009[3] and is required to respond to the RAI within 30 days of receipt. UniStar Nuclear Energy is working closely with AREVA and Bechtel on these questions concerning the interaction of non-seismic structures with seismic Category I structures. The response schedule for answering this RAI No. 65 question will be provided shortly after the AREVA response.

COLA Impact

None

RAI No. 65

Question 03.07.02-20

In FSAR Section 3.7.2.8 on page 3.0-42, it states that the existing non-seismic bulkhead could potentially interact with the Ultimate Heat Sink (UHS) Makeup Water Intake Structure and UHS Electrical Building. Identify and describe the methods used to determine that this structure will not have any unacceptable interaction with either of the Seismic Category I structures?

Response

In the original configuration (shown in the Figure 1(a)), the Seismic Category I Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS) and Seismic Category I UHS Electrical Building (EB) were to be located close to the existing non-seismic bulkhead, and there was a potential for interaction between these structures and the bulkhead. UniStar letter[4] dated 1/14/2009 informed the NRC of a configuration change to relocate UHS MWIS and UHS EB structures approximately 700 ft South-East (with respect to CCNPP Unit 3 plant coordinate system) from the Unit 1 and 2 forebay. The relocation of UHS MWIS and UHS EB are shown in the Figure 1(b) below. Due to the relocation of the UHS MWIS and UHS EB structures, there is no possibility of interaction between the existing non-seismic bulkhead and these Seismic Category 1 structures.

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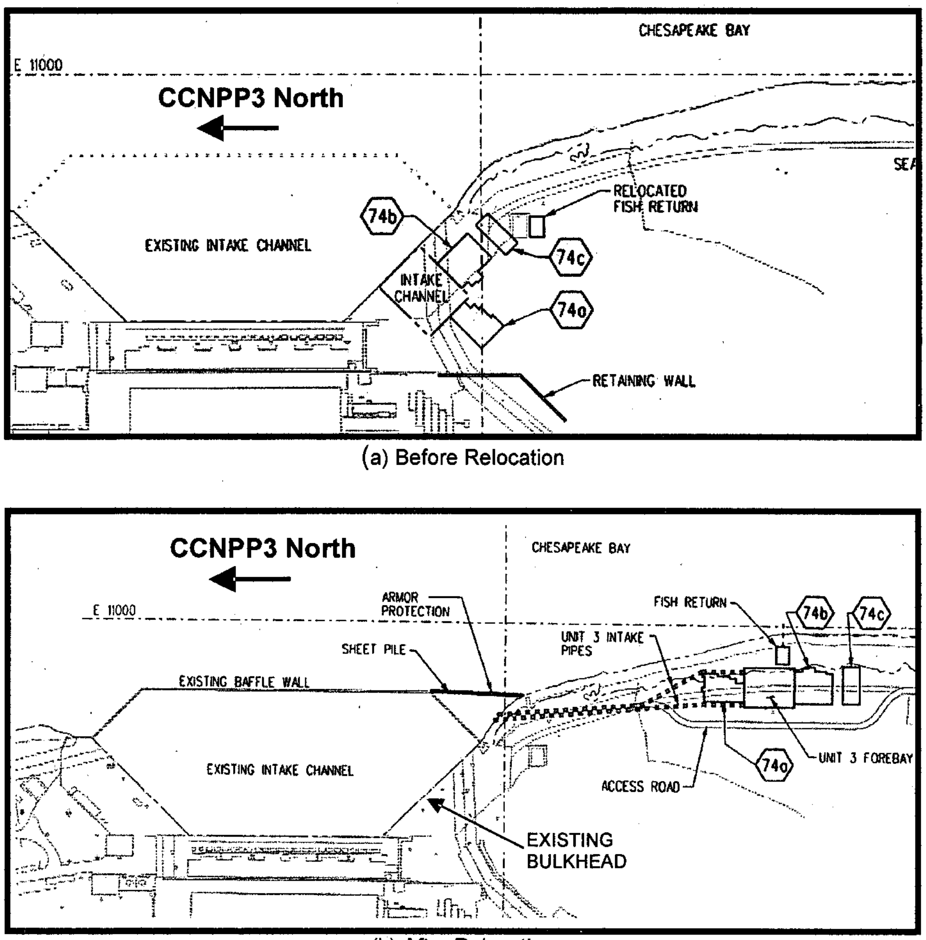
[4]UniStar Nuclear Energy Letter UN#09-005, from Greg Gibson (UniStar Nuclear Energy) to Document Control Desk, U.S. NRC, Intake Structure Relocation Changes for Environment Report, dated January 14, 2009



74a – Circulating Water Intake Structure
74b – UHS Makeup Water Intake Structure
74c – UHS Electrical Building

Figure 1: Layout of CCNPP3 Site-Specific Structures Before and After Relocation

COLA Impact

None

RAI No. 65

Question 03.07.02-23

At the end of FSAR Section 3.7.2.15, on page 3.0-44, there is a description of a comparison of an analysis result using ANSYS to solve the complex eigen-value solution of the non-classical damping formulation with an analysis result using GT STRUDL to solve the real eigen-value solution of the classical damping formulation in which the off-diagonal terms of the damping matrix are neglected. It is not clear from the discussion which of the damping methods was used in the seismic analysis of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS). In addition, no comparison of the results using the two methods cited has been provided. Provide the method used to account for damping in the seismic analysis of the UHS MWIS and provide in the FSAR the results of the study comparing the non-classical damping formulation with the classical damping formulation.

Response

1. Composite modal damping ratios were calculated using GT STRUDL (v.29.1) by neglecting the off-diagonal terms from the damping matrix ($\zeta_i = \{\Phi_j\}^T[C]\{\Phi_j\}/2\omega_j$). These composite modal damping ratios (shown in Table 2 using 7% structural damping) were used in the modal superposition time history analysis of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS). As noted in FSAR Section 3.7.2.15, composite modal damping ratios were conservatively capped at 15%.

2. For verification and to investigate accuracy of the approach described above, a study was conducted where modal damping ratios were calculated by complex eigensolution of the non-classically damped problem using ANSYS (v.11) and compared with the damping ratios computed by GTSTRUDL (v 29.1). A comparison of the modal damping ratios by the two approaches is presented in Table 1. Table 1 shows six modes (associated with rigid body motion) for the six soil analysis cases described in FSAR Section 3.7.2.4. The depicted modes represent only the soil driven modes and account for a cumulative mass participation greater than 90%. Zero percent (0%) structural damping was used to calculate the composite modal damping ratios shown in the comparison. Since composite modal damping ratios were capped at 15%, the structural response was not affected by the structural damping being used. This is further illustrated in 3. and 4. of this response. The stiffness and mass proportional factors in the Rayleigh damping formulation (ASCE 4-98, Eq. 3.1-2) were set to zero (i.e., α=0 and β=0). Table 1 depicts reasonable agreement between the ANSYS (v.11) and GT STRUDL (v.29.1) results.

Table 1, Comparison of modal damping ratios (ζ) calculated using composite modal damping formulation (GT STRUDL v.29.1) and complex eigensolution (ANSYS v.11)

	Soil analysis cases with no embedment						Soil analysis cases with embedment					
	50% G		100% G		200% G		50% G		100% G		200% G	
Mode	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ
1	0.171	0.175	0.165	0.169	0.150	0.148	0.158	0.159	0.140	0.139	0.114	0.114
2	0.175	0.175	0.166	0.165	0.155	0.158	0.155	0.154	0.148	0.150	0.131	0.133
3	0.517	0.507	0.512	0.502	0.504	0.489	0.437	0.431	0.432	0.426	0.423	0.413
4	0.090	0.094	0.082	0.084	0.069	0.069	0.054	0.056	0.043	0.045	0.029	0.031
5	0.344	0.340	0.336	0.334	0.317	0.314	0.202	0.201	0.186	0.185	0.145	0.144
6	0.336	0.328	0.332	0.325	0.325	0.310	0.205	0.202	0.204	0.193	NA[1]	NA[1]
7	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	0.208	0.193

1. This is a structural mode, not a soil mode.

During the NRC onsite Technical Audit held in Frederick, MD offices between March 26 and March 29, 2009, the NRC staff requested information about the structural damping used to calculate structural responses and generate in-structure response spectra (ISRS) for the MWIS. The requested information is provided below.

3. Composite modal damping ratios (described in 1.) to calculate the structural response as well as to generate ISRS reported in FSAR Figures 3.7-39 through 3.7-41, were based on seven percent (7%) SSE level structural damping. Even though stress levels in the structural elements are low, use of 7% structural damping to generate ISRS was reasonable because the increase in the damping coefficients of the impedance functions due to embedment was neglected, as stated in FSAR Section 3.7.2.4.

4. To validate the use of 7% SSE level damping for the generation of ISRS, additional studies based on 4% OBE level structural damping were conducted. The additional studies accounted for increased soil damping coefficients due to embedment effects. Table 2 shows the comparison of the composite modal damping ratios for 60 modes using 7% structural damping with those using 4% structural damping. The additional studies utilized the following approach to calculate the composite modal damping ratios of soil driven modes and structural modes:

    a. The modal damping ratios associated with soil driven modes were calculated using 0% structural damping.

    b. The modal damping ratios for structural modes were set to 4% in accordance with NRC RG 1.61, Rev. 1.

    c. The composite modal damping ratios for soil analysis cases with embedment were set equal to that of cases without embedment.

    d. The modal damping ratios of soil driven modes were capped at 15%.

ISRS were generated using the modal damping ratios corresponding to 4% structural damping and were compared with those using 7% structural damping. Though modal damping ratios using 4% structural damping differ occasionally from those using 7% structural damping, results from the additional studies demonstrate that the ISRS reported in FSAR Figures 3.7-39 through 3.7-41 are unaffected.

Table 2, Comparison of modal damping ratios (ζ) using 7% and 4% structural damping

MODE	Soil analysis cases using 7% structural damping						Soil analysis cases using 4% structural damping					
	Without embedment			With embedment			Without embedment			With embedment		
	0.5G ζ	1.0G ζ	2.0G ζ	0.5G ζ	1.0G ζ	2.0G ζ	0.5G ζ	1.0G ζ	2.0G ζ	0.5G ζ	1.0G ζ	2.0G ζ
1	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.148	0.150	0.150	0.148
2	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150
3	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150
4	0.115	0.111	0.098	0.079[3]	0.072[3]	0.063[3]	0.094	0.084	0.069	0.094	0.084	0.069
5	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150	0.150
6	0.150	0.150	0.150	0.150	0.150	0.042	0.150	0.150	0.150	0.150	0.150	0.040
7	NA[2]	0.041	0.048	NA[2]	0.048	0.150	0.040	0.040	0.040	0.040	0.040	0.150
8	NA[2]	0.079	0.050	NA[2]	0.044	0.030	0.040	0.040	0.040	0.040	0.040	0.040
9	NA[2]	0.056	0.105	NA[2]	0.092	0.033	0.040	0.040	0.040	0.040	0.040	0.040
10	NA[2]	0.054	0.049	NA[2]	0.047	0.034	0.040	0.040	0.040	0.040	0.040	0.040
11	NA[2]	0.055	0.049	NA[2]	0.048	0.034	0.040	0.040	0.040	0.040	0.040	0.040
12	NA[2]	0.056	0.050	NA[2]	0.049	0.129	0.040	0.040	0.040	0.040	0.040	0.040
13	NA[2]	0.150	0.150	NA[2]	0.129	0.111	0.040	0.040	0.040	0.040	0.040	0.040
14	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.090	0.040	0.040	0.040	0.040	0.040	0.040
15	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.062	0.040	0.040	0.040	0.040	0.040	0.040
16	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.061	0.040	0.040	0.040	0.040	0.040	0.040
17	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.064	0.040	0.040	0.040	0.040	0.040	0.040
18	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.076	0.040	0.040	0.040	0.040	0.040	0.040
19	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.076	0.040	0.040	0.040	0.040	0.040	0.040
20	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.072	0.040	0.040	0.040	0.040	0.040	0.040
21	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.084	0.040	0.040	0.040	0.040	0.040	0.040
22-60	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	NA[2]	0.040	0.040	0.040	0.040	0.040	0.040

2 Modes not used during modal superposition time history analysis. The selected modes were based on inclusion of 99.9% cumulative modal mass in each orthogonal direction as shown in FSAR Tables 3.7-7 through 3.7-12.

3 For soil analysis cases using 7% structural damping, the composite modal damping ratio of soil driven modes for soil analysis cases with embedment is less than that of soil driven modes for soil analysis cases without embedment. The reason lies in the fact that soil spring stiffness is increased to reflect the embedment, whereas the corresponding permitted increase in the damping coefficients is neglected, as described in FSAR Section 3.7.2.4 (page 3-36).

COLA Impact

Part 2, FSAR of the CCNPP Unit 3 COLA will be updated in a future COLA revision to incorporate the changes to FSAR Sections 3.7.2.15 that are indicated below, and also add Table 3.7-13:

3.7.2.15 Analysis Procedure for Damping

For the site-specific Seismic Category I, UHS Makeup Water Intake Structure, Rayleigh damping mass proportional factors and Rayleigh damping stiffness proportional factors are calculated for structural frequencies associated with each of the six soil cases identified in Section 3.7.2.4.

For the soil, damping coefficients are generated per ASCE 4-98 (ASCE, 1986)., ~~incorporating appropriate stiffness proportional factors~~. As discussed in Section 3.7.2.4, the beneficial effect of embedment is ignored during the calculation of soil damping. Calculated ~~stiffness~~ damping is lumped for the whole foundation. Subsequently, the ~~stiffness~~ damping is distributed based on tributary area.

All SSI problems are non-classical damping problems in nature, because the system consists of two subsystems (i.e., structure and soil) with significant variation in damping. Non-classical damping means the multiplication of the eigenmatrix φ^T and system damping matrix (C) is a fully-populated matrix. Thus, modal differential equations are coupled and classical modal decomposition is no longer valid.

This consideration of "Composite Modal Damping" is a method of approximating a non-classical damping problem with a classical damping problem. For this method, the diagonal terms of $\varphi^T C \varphi$ are retained, with the off-diagonal terms neglected, such that classical mode decomposition is preserved. However, such an approximation may, in certain cases, yield results which are inaccurate beyond acceptable bounds.

To investigate the accuracy of the composite modal damping methodology for the structure and soil subsystems of the UHS Makeup Water Intake Structure, composite modal damping ratios are calculated per two different approaches, and associated finite element programs:

- ANSYS (v.11) complex eigensolution of the non-classical (or non-proportional) damping formulation.

- GT STRUDL (v.29.1) real eigensolution of the classical (or proportional) damping formulation.

For soil driven modes, close correlation is realized between the two approaches as show in Table 3.7-13. The comparison of modal damping ratios is based on the use of zero percent structural damping. Composite modal damping ratios calculated by GT STRUDL (v.29.1) are used in the modal superposition time history analysis of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS). To retain conservatism, composite modal damping is capped at 15 percent.}

Add new Table 3.7-13 as indicated below:

Table 3.7-13, Comparison of modal damping ratios (ζ) calculated using composite modal damping formulation (GT STRUDL v.29.1) and complex eigensolution (ANSYS v.11)

	Soil analysis cases with no embedment						Soil analysis cases with embedment					
	50% G		100% G		200% G		50% G		100% G		200% G	
Mode	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ	Ansys ζ	GT ζ
1	0.171	0.175	0.165	0.169	0.150	0.148	0.158	0.159	0.140	0.139	0.114	0.114
2	0.175	0.175	0.166	0.165	0.155	0.158	0.155	0.154	0.148	0.150	0.131	0.133
3	0.517	0.507	0.512	0.502	0.504	0.489	0.437	0.431	0.432	0.426	0.423	0.413
4	0.090	0.094	0.082	0.084	0.069	0.069	0.054	0.056	0.043	0.045	0.029	0.031
5	0.344	0.340	0.336	0.334	0.317	0.314	0.202	0.201	0.186	0.185	0.145	0.144
6	0.336	0.328	0.332	0.325	0.325	0.310	0.205	0.202	0.204	0.193	NA[1]	NA[1]
7	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	NA[1]	0.208	0.193

1. This is a structural mode, not a soil mode.

RAI No. 65

Question 03.07.02-26

SRP 3.7.2, SRP Acceptance Criteria 14 states that the determination of seismic overturning moments and sliding forces should include three components of input motion and conservative consideration of the simultaneous action of the vertical and horizontal seismic forces. How overturning moments and sliding forces are determined has not been provided in either FSAR Section 3.7.2, 3.8.5 or in Section 3E.4. The applicant is requested to provide this information in Section 3.7.2 and describe how this information is used in determining the overturning and sliding stability of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure and UHS Electrical Building.

Response

FSAR Section 3.7.2.14 refers to Section 3.8.5 for determination of seismic stability of the Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS) and UHS Electrical Building (EB). FSAR Section 3.8.5.5.4 provides the calculated and allowable factors of safety against overturning and sliding for UHS MWIS by referring to Table 3.8-1, and redirects to Appendix 3E, Section 3E.4 for the details of the stability analyses. FSAR Appendix 3E, Sections 3E.4.1 and 3E.4.2 discuss the applicable loading conditions that are considered for determination of seismic stability for UHS MWIS and EB, respectively. These sections also document the calculated factors of safety against overturning, sliding and flotation for critical loading conditions.

Overturning and sliding is not applicable for UHS EB because the structure is essentially completely embedded in soil. Therefore, the response given below pertains to the methodology adopted for determining seismic stability for the UHS MWIS.

<u>How overturning moments and sliding forces are determined for UHS MWIS:</u>

An equivalent static analysis is performed using the design accelerations from the time history analysis as documented in FSAR Section 3.7.2.1.4. To determine seismic stability, the overturning moments and sliding forces are determined using GT STRUDL. The three components of ground motion are considered, in accordance with SRP 3.7.2, Acceptance Criteria 14, by using the 100-40-40 combination rule described in FSAR Section 3.7.2.6.

For each of the twenty four (24) co-directional response combinations, the overturning moments are determined about each of the four edges of the basemat for seismic forces, static and dynamic lateral earth pressures, and hydrostatic and hydrodynamic forces. The results about each edge are enveloped to compute the maximum overturning moments. The sliding forces in the two horizontal directions are also determined for the above mentioned co-directional response combinations, and the absolute maximum sliding force in each direction is computed from these results. The total sliding force is then calculated using vectorial sum of these horizontal components.

How the information is used to determine overturning and sliding stability for UHS MWIS:

The restoring moments about each edge are computed using the self weight of the structure, weight of the permanent equipment and contained water during normal operation, 25% of the design live load and 75% of the design snow load, per SRP 3.7.2, Acceptance Criteria 3D. The factor of safety against overturning is then calculated as the ratio of the restoring and overturning moment. The sliding resistance is provided by the friction at the bottom of the basemat and the passive resistance of soil against shear keys (see FSAR Section 3.8.5.5). The factor of safety against sliding is calculated as the ratio of resisting force and the total sliding force.

The stability evaluation is performed for stability load combinations provided in SRP 3.8.5, Acceptance Criteria 3 (see FSAR Table 3E.4-1, Load Cases # 6 to 9).

The stability evaluation based on the equivalent static analysis is conservative since it ignores the dynamic nature of seismic inertia loading in both horizontal and vertical directions. Under earthquake excitation, large responses occur for only a fraction of second in each cycle, and are not sustained. Therefore, the factors of safety for overturning and sliding that are based on equivalent static method are conservative. The hydrodynamic pressures due to contained water, which are used in the stability evaluation, are calculated using a design acceleration of 0.5g rather than the 0.35g acceleration determined from the time history analysis. Therefore, the stability analysis meets SRP 3.7.2, Acceptance Criteria 14B requirement for conservatism.

COLA Impact

Part 2, FSAR of the CCNPP Unit 3 COLA will be updated in a future COLA revision to incorporate the changes to FSAR Sections 3.7.2.14 and 3.7.2.16 that are identified below:

3.7.2.14 Determination of Dynamic Stability of Seismic Category I Structures

~~Refer to Section 3.8.5 for specific details related to both overturning and sliding stability for the UHS Makeup Water Intake Structure and UHS Electrical Building for the extreme environment SSE, Probable Maximum Hurricane (PMH), and tornado events.~~

Refer to Section 3.8.5 and Appendix 3E.4 for specific details related to both overturning and sliding stability for the UHS Makeup Water Intake Structure and UHS Electrical Building for the extreme environment SSE, Probable Maximum Hurricane (PMH), and tornado events.

UHS Makeup Water Intake Structure

The stability of the UHS Makeup Water Intake Structure for applicable loading is determined using the stability load combinations provided in SRP 3.8.5, Acceptance Criteria 3 (NRC, 2007a), listed as Load Combinations 6 to 9 in FSAR Table 3E.4-1.

For determination of seismic stability, the overturning moments about each of the four edges of the basemat and sliding forces at the bottom of the basemat are computed by using the results from the equivalent static analysis. These responses include the effects of seismic forces, static and dynamic lateral earth pressures, and hydrostatic and hydrodynamic forces. The effect of three components of ground motion is combined by using the 100-40-40 combination rule described in FSAR Section 3.7.2.6. The analysis results from the twenty four (24) co-directional response combinations are enveloped to compute the maximum overturning moments about the edges of the basemat, and the sliding forces in the horizontal directions. The maximum resultant sliding force is determined from the vectorial sum of the maximum sliding forces in the two horizontal directions.

The restoring moments due to the self weight of the structure, weight of the permanent equipment and contained water during normal operation, 25% of the design live load and 75% of the design snow load are also determined from the equivalent static analysis. The sliding resistance is calculated manually and includes the effect of friction at the bottom of the basemat and the passive resistance of soil against the shear keys. Factors of safety against overturning and sliding for the UHS Makeup Water Intake Structure are documented in FSAR Table 3.8-1.

UHS Electrical Building

Since UHS Electrical Building is essentially completely embedded, seismic stability evaluation is not applicable.

3.7.2.16 References

ACE, 2003. Engineering and Design - Time History Dynamic Analysis of Concrete Hydraulic Structures, EM-1110-2-6051, U.S. Army Corps of Engineers Manual, December 2003.

ACI, 2006. Seismic Design of Liquid-Containing Concrete Structures, ACI 350.3-06, American Concrete Institute, 2006.

ASCE, 1986. Seismic Analysis of Safety-Related Nuclear Structures and Commentary, ASCE Standard 4098, American Society of Civil Engineers, September 1986.

NRC, 1973. Design Response Spectra for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.60, U.S. Nuclear Regulatory Commission, December 1973.

NRC, 1978. Development of Floor Design Response Spectra for Seismic Design of Floor-Supported equipment or Components, Regulatory Guide 1.122, U.S. Nuclear Regulatory commission, February, 1978.

NRC, 2007. Fire Protection for Nuclear Power Plants, Regulatory Guide 1.189, Revision 1, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2007a. Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants, NUREG-0800, Section 3.8.5, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2008. Earthquake Engineering Criteria for Nuclear Power Plants, Title 10, Code of Federal Regulations, Part 50, Appendix S, U. S. Nuclear Regulatory Commission, February 2008.}

Enclosure 3

Enlarged tables associated with RAI No. 58 - Seismic Design Parameters, Question 03.07.02-8,
Table 1, Summary of Moment Capacities and Demands for Structural Elements of Intake Structure (IS) and Pumphouse (PH) and
Table 2, Summary of Shear Capacities and Demands for Structural Elements of Intake Structure (IS) and Pumphouse (PH)
Calvert Cliffs Nuclear Power Plant Unit 3

Table 1, Summary of moment capacities and demands for structural elements of Intake Structure (IS) and Pumphouse (PH)

Structural Elements of IS & PH	h thickness (ft)	y_t=h/2 half thickness (ft)	b unit width (ft)	$I_g=b*h^3/12$ gross moment of inertia, (ft^4)	[4, 5] $M_{cr} = f_r*I_g/y_t$ cracking moment, Eq. (9-8) of ACI 349-01 (kip-ft)	[3] M_{xx} out-of-plane bending moment, (kip-ft)	[3] M_{yy} out-of-plane bending moment, (kip-ft)	[3] M_{xy} normal twisting moment, (kip-ft)	[4] M_a maximum moment demand, (kip-ft) $=\lvert M_{xx}\rvert+\lvert M_{xy}\rvert$ or $=\lvert M_{yy}\rvert+\lvert M_{xy}\rvert$	M_{cr}/M_a
[1] Side Wall of IS (N-S direction)	4.0	2.0	1.0	5.3	203	40	167	6	173	1.17
						36	180	8	188	1.08
[1a] Operating Deck of IS	3.0	1.5	1.0	2.3	114	50	24	4	54	2.11
[2] Back Wall of IS (E-W direction)	4.0	2.0	1.0	5.3	203	146	80	14	160	1.26
[2] Interior Wall of IS	4.0	2.0	1.0	5.3	203	36	64	15	79	2.56
[2] Side Wall of PH (N-S direction)	2.0	1.0	1.0	0.7	51	12	17	8	25	2.04
[2] Roof Slab of PH	2.0	1.0	1.0	0.7	51	20	12	4	24	2.12
[2] Front & Back Wall of PH (E-W direction)	2.0	1.0	1.0	0.7	51	10	7	2	12	4.25

1 Maximum moment resultants are interpolated from the GTSTRUDL finite element contours at the location 'A' of Figure 1. The location 'A' represents the intersection of the side wall and the foundation basemat (4 feet thick). It is to be noted that the results presented in FSAR Table 3E.4-2 for the side wall represent enveloped results at different locations. The results in the Table 3E.4-2 at Joint 5217 correspond to the GTSTRUDL model centerline of foundation basemat. The interpolated location 'A' in this RAI response is 2 feet above Joint 5217. The values in the top row correspond to load combination DL + LL + H (factored combination including wave pressure loading) divided by load factor of 1.5; whereas the values in the bottom row correspond to the load combination DL + LL + H + SSE (seismic combination without wave pressure loading) divided by a load factor of 1.0 at the same joint 5217.

1a Maximum moment resultants are interpolated from the GTSTRUDL finite element contour values at the location 'B' of Figure 1. The location 'B' represents the intersection of the operating deck and the face of the back wall. The values correspond to governing load combinations.

2 Enveloped moment resultants at different locations and load combinations from the GTSTRUDL finite element contour values are considered.

3 Moments are from governing load combinations.

4 Definitions of M_{cr} and M_a based on ACI-349-01 Section 9.0.

5 The modulus of rupture of concrete, f_r = 530 psi, is calculated as per ACI-349-01 Eq. (9-9) based on 5000 psi concrete strength.

Table 2, Summary of shear capacities and demands for structural elements of Intake Structure (IS) and Pumphouse (PH)

Structural Elements of IS & PH	h thickness (ft)	[3] d distance (ft)	b unit width (ft)	A_g=b*h gross area of section, (ft^2)	[4] Max (N_{xx}, N_{yy}) in-plane factored tension force (kip)	[5] V_{u_ip} = N_{xy} in-plane factored shear force (kip)	[6] V_{u_oop} = Max (V_{xx}, V_{yy}) out-of-plane factored shear force (kip)	[7, 10] V_{c_ip} in-plane shear strength (kip)	[8, 10] V_{c_oop} out-of-plane shear strength (kip)	[9] $\frac{V_{c\_ip}}{V_{u\_ip}}$ in-plane shear ratio	[9] $\frac{V_{c\_oop}}{V_{u\_oop}}$ out-of-plane shear ratio
[1] Side Wall of IS (N-S direction)	4.0	3.57	1.0	4.0	0	20	70	122	73	6.10	1.04
					0	28	51		73	4.36	1.42
[1a] Operating Deck of IS	3.0	2.57	1.0	3.0	8	18	15	92	50	5.09	3.36
[2] Back Wall of IS (E-W direction)	4.0	3.57	1.0	4.0	44	30	35	122	62	4.07	1.76
[2] Interior Wall of IS	4.0	3.57	1.0	4.0	62	39	17	122	57	3.13	3.35
[2] Side Wall of PH (N-S direction)	2.0	1.67	1.0	2.0	60	29	6	61	20	2.10	3.30
[2] Roof Slab of PH	2.0	1.67	1.0	2.0	33	13	6	61	26	4.70	4.47
[2] Front & Back Wall of PH (E-W direction)	2.0	1.67	1.0	2.0	8	18	5	61	32	3.39	6.57

1 Maximum shear force resultants are interpolated from the GTSTRUDL finite element contour values at the location 'A' of Figure 1. The location 'A' represents the intersection of the side wall and the foundation basemat (4 feet thick). It is to be noted that the results presented in FSAR Table 3E.4-2 for the side wall represent enveloped results at different locations. The results in the Table 3E.4-2 at Joint 5217 correspond to the GTSTRUDL model centerline of foundation basemat. The interpolated location 'A' in this RAI response is 2 feet above Joint 5217. The values in the top row correspond to load combination DL + LL + H (factored combination including wave pressure loading); whereas the values in the bottom row correspond to the load combination DL + LL + H + SSE (seismic combination without wave pressure loading) at the same joint 5217.

1a Maximum shear force resultants are interpolated from the GTSTRUDL finite element contour values at the location 'B' of Figure 1. The location 'B' represents the intersection of the operating deck and the face of the back wall. The values correspond to governing load combinations.

2 Enveloped shear resultants at different locations and load combinations from the GTSTRUDL finite element contour values are considered.

3 Distance from extreme compression fiber to centroid of longitudinal tension reinforcement.

4 Maximum enveloped in-plane factored tension force (Nxx, Nyy) from the GTSTRUDL finite element contour values is considered and used as the input (Nu) in Eq. 11-8 of ACI-349-01.

5 Enveloped in-plane factored shear force (Nxy) from the GTSTRUDL finite element contour values.

6 Enveloped out-of-plane factored shear force (Vxx, Vyy) from the GTSTRUDL finite element contour values is considered.

7 Equation (21-7) of ACI-349-01 is used to determine the concrete nominal shear strength for in-plane shear with ρn = 0 and αc =3.0.

8 Equation (11-8) of ACI-349-01 is used to determine the concrete shear strength for out-of-plane shear for wall sections subject to axial tension.

9 The wall/slab is un-cracked when Vc/Vu > 1.0 per Table 3-1 'Effective Stiffness of Reinforced Concrete Members' of ASCE/SEI 43-05.

10 The concrete strength is 5000 psi.