Greg Gibson
Vice President, Regulatory Affairs

750 East Pratt Street, Suite 1600
Baltimore, Maryland 21202

UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

July 23, 2010

UN#10-193

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. 144, Other Seismic Category I Structures, and RAI No. 145, Foundations

References:

1) Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI No. 144 SEB 2196" email dated August 28, 2009

2) Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI No 145 SEB 2197" email dated August 28, 2009

3) UniStar Nuclear Energy Letter UN#10-109 from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 144, Other Seismic Category I Structures and RAI No. 145, Foundations, dated April 8, 2010.

4) UniStar Nuclear Energy Letter UN#09-390 from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 144, Other Seismic Category I Structures and RAI No. 145, Foundations, dated September 28, 2009.

5) UniStar Nuclear Energy Letter UN#10-074 from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, Impact of U.S. EPR FSAR RAI Responses on CCNPP Unit 3 FSAR Section 3.7, dated March 15, 2010.

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6) UniStar Nuclear Energy Letter UN#09-427 from Greg Gibson to Document Control Desk, U.S. NRC, Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5, dated October 9, 2009.

The purpose of this letter is to respond to the requests for additional information (RAIs) 144 and 145 identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated August 28, 2009 (Reference 1 and Reference 2). These RAIs address "Other Seismic Category I Structures" and "Foundations," as discussed in Section 3.8 of the Final Safety Analysis Report (FSAR), as submitted in Part 2 of the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 Combined License Application (COLA), Revision 6.

Reference 3 anticipated that the responses to RAI 144 and RAI 145 would be provided to the NRC by July 23, 2010.

This letter contains six enclosures:

- Enclosure 1 provides responses to RAI 144, Questions 03.08.04-01 through 03.08.04-14. The responses to Question 03.08.04-02 Part 2 and Question 03.08.04-05 previously provided in Reference 4 have been included and updated.

- Enclosure 2 provides responses to RAI 145, Questions 03.08.05-01 through 03.08.05-06, except Question 03.08.05-02 and the Nuclear Island (NI) portions of Question 03.08.05-04. The response to these questions will be affected by the seismic reconciliation of the CCNPP Unit 3 FSAR with the finite element model of the NI being prepared for the U.S. EPR FSAR. The response to these questions will be provided concurrent with the other updates to FSAR Sections 3.7 and 3.8, as discussed in Reference 5, by September 21, 2010.

- Enclosure 3 provides FSAR Section 3.8 in its entirety with new text shown in red underline. To improve readability, the deleted or "strike out" text is not shown.

- Enclosure 4 provides an update to COLA Part 7 "Departures and Exemption Requests." The departure on tilt of the Emergency Power Generating Building (EPGBs), and Essential Service Water Buildings (ESWBs) has been updated to reflect the result of the settlement analysis in updated CCNPP Unit 3 FSAR Section 2.5.4 as provided in Reference 6. In addition, a new departure is necessary because the coefficient of static friction below the structures is less than the 0.7 specified in the U.S. EPR FSAR.

- Enclosure 5 provides an update to COLA Part 10 "Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) and ITAAC Closure."

- Enclosure 6 provides conforming changes to other FSAR sections and COLA Parts.

Enclosures 3, 4, 5, and 6 present revised COLA content. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

GTG/SJS/mdf

This letter does not contain any sensitive or proprietary information and does not include any new regulatory commitments

If there are any questions regarding this transmittal, please contact me at (410) 470-4205, or Mr. Wayne A. Massie at (410) 470-5503.

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

Executed on July 23, 2010

Greg Gibson

Enclosures:

1) Response to NRC Request for Additional Information, RAI 144, Other Seismic Category I Structures, Questions 03.08.04-01 through 03.08.04-14.
2) Response to NRC Request for Additional Information RAI 145, Foundations, Questions 03.08.05-01, 03, 04, 05 and 06.
3) Markup of FSAR Section 3.8, Design of Category I Structures, Calvert Cliffs Nuclear Power Plant, Unit 3.
4) Markup of COLA Part 7, Departures and Exemption Requests, Calvert Cliffs Nuclear Power Plant, Unit 3.
5) Markup of COLA Part 10 "Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) and ITAAC Closure."
6) Conforming Changes to other FSAR Sections and COLA Parts.

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
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/SJS/mdf

Enclosure 1

Response to NRC Request for Additional Information

RAI 144, Other Seismic Category I Structures,
Questions 03.08.04-01 through 03.08.04-14,

Calvert Cliffs Nuclear Power Plant, Unit 3

RAI 144

Question 03.08.04-1

Calvert Cliffs Unit 3 FSAR Section 3.8.4.1 provides a description of Seismic Category I Structures other than containment and containment internal structures. Address the following items related to these structures.

1. FSAR Section 3.8.4.1 states:

   The U.S. EPR FSAR includes the following COL Items in Section 3.8.4:

   > A COL applicant that references the U.S. EPR design certification will describe any differences between the standard plant layout and design of Seismic Category I structures required for site-specific conditions.
   >
   > A COL applicant that references the U.S. EPR design certification will address site-specific Seismic Category I structures that are not described in this section.

   The Calvert Cliffs Unit 3 FSAR provided information to address the second COL item but not the first. The applicant is requested to address the first COL item as well, or state in the FSAR (as was done for the other items) that "No departures or supplements" apply.

2. FSAR Section 3.8.4.1 lists several site-specific Seismic Category I structures, including the UHS Makeup Water Intake Structure. However, FSAR Figure 9.2-4 - UHS Makeup Water and CW Intake Structures, shows several structural features which are not included in the list of site-specific Seismic Category I structures in FSAR Section 3.8.4.1. Identify whether the structural features listed below (obtained from Figure 9.2-4) are considered as Seismic Category I, and if not, explain why.

   a. Existing Bulkhead
   b. New Sheet Pile Bulkhead
   c. New Channel Wall
   d. New Dredged Intake Channel

   Identify where all of these items are listed in FSAR Table 3.2-1. If these items are considered as Seismic Category I or II, identify where the design and analysis descriptions are provided.

3. FSAR Figure 9.2-4 shows that the CW Intake Structure is quite close to the UHS Makeup Water Intake Structure. According to FSAR Table 3.2-1, the CW Intake Structure is classified as Seismic Category "CS" which means Conventional Seismic. Explain why this structure isn't classified as Seismic Category II since it appears that consideration of potential seismic interaction effects with the adjacent UHS Makeup Water Intake Structure is needed.

4. The EPR FSAR and the CCNPP Unit 3 FSAR do not provide a description of the analysis and design results for the radwaste structures consisting of the Nuclear Auxiliary Building (NAB) and the Radioactive Waste Processing Building (RWPB). Explain where this information is located. Similarly, where is the description of the analysis and design results for Seismic Category II structures?

Response

Part 1

For Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3, the standard plant layout and design of standard Seismic Category I Structures is as described in the U.S. EPR FSAR without departures. This statement has been added to CCNPP Unit 3 Final Safety Analysis Report (FSAR) Section 3.8.4.1 as shown in Enclosure 3.

Part 2

Due to relocation of intake structures as shown in CCNPP Unit 3 FSAR Revision 6, structural features in the vicinity of the Ultimate Heat Sink (UHS) Makeup Water and Circulating Water (CW) Intake Structures listed in the question are not applicable to the new location of these structures. As shown on CCNPP Unit 3 FSAR Figures 2.4-51 and 9.2-4, the following structures are in the vicinity of the Seismic Category I UHS Makeup Water Intake Structure, UHS Electrical Building and buried Intake Pipes:

1. CCNPP Units 1 & 2 Baffle Wall
2. Sheet Pile Wall
3. Circulating Water System (CWS) Makeup Water Intake Structure (MWIS)
4. Forebay

The safety and seismic classifications of these structures, except the Baffle Wall, are provided in CCNPP Unit 3 FSAR Table 3.2-1 in the groups called "UHS Makeup Water System" and "Circulating Water System." The following paragraphs provide discussion on each of these structures.

Baffle Wall and Sheet Pile Wall: As shown in CCNPP Unit 3 FSAR Figure 2.4-51, the Baffle Wall for CCNPP Units 1 and 2 is an existing sheet pile wall with three sides. The Baffle Wall is non-seismic and consists of sheet pile panels supported by pile bents. The panels are about 32 ft tall and extend from 4.5 ft above mean sea level to about 23 feet above the inlet bed, permitting bottom water of the Chesapeake Bay to enter the inlet area. The Sheet Pile Wall called out in CCNPP Unit 3 FSAR Figure 2.4-51 is a new wall which is similar to the existing Baffle Wall but extends to the bed of the bay. None of these walls perform any safety-related function and thus are not classified as Seismic Category I. The updated CCNPP Unit 3 FSAR Section 3.7.2.8[1] describes potential interaction of these structural elements with Seismic Category I Intake Pipes.

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[1] UN#09-519, UniStar Nuclear Energy letter from Greg Gibson to Document Control Desk, U.S. NRC, Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Section 3.7 and response to FSAR Section 3.7 RAI sets 19, 25, 58, 63, 65, 112, 113, 139, 158, 159, 167, 168, 179, 180, 181, and 193, dated December 29, 2009.

<u>CWS MWIS</u>: This structure provides makeup water for the circulating water system, which is a non-safety related system. Therefore, the structure is not classified as Seismic Category I. As shown in updated CCNPP Unit 3 FSAR Figures 10.4-4 and 10.4-5[2], due to at least 11 feet of soil between the embedded walls of this structure and buried Intake Pipes, the structure does not have potential to interact with the Intake Pipes. Due to a common foundation basemat shared with the UHS MWIS and Forebay, and integrally connected walls with the Forebay, the concrete portion of the CWS MWIS is designed to the same seismic criteria as the UHS MWIS. The updated CCNPP Unit 3 FSAR Section 3.7.2.8[1] discusses interaction of CWS MWIS with safety-related structures.

The upper portion of the CWS MWIS, known as the Pump House Enclosure, is comprised of structural steel. The height of this enclosure is about 30 ft above grade and it is about 50 feet away from the south end of safety-related buried Intake Pipes in the Forebay and about 150 feet away from UHS MWIS. Hence, there is no potential for adverse interaction of the Pump House Enclosure with any Seismic Category I structure.

Any elements of the Baffle Wall, Sheet Pile Wall or Pump House Enclosure, which could become tornado generated missiles, are enveloped by the missile design spectrum described in CCNPP Unit 3 FSAR Section 3.5.1.4. The safety-related structures are designed for these missiles.

<u>Forebay</u>: The Forebay is positioned between the CWS MWIS and the safety-related Intake Pipes on the north side and the safety-related UHS MWIS on the south side. The Forebay has a dual function of supplying make-up water to the non-safety related Circulating Water Makeup System and to the safety-related UHS Makeup Water System. Since the Forebay is an extension of the safety-related Intake Pipes as the transportation route of makeup water for the safety-related UHS MWIS, its seismic classification was changed to Seismic Category I in response to RAI 182[2]. The Forebay is analyzed and designed to the same requirements as the UHS MWIS.

<u>Design and Analysis Descriptions of Seismic Category I and II Structures</u>

The description of analysis and design of the Seismic Category I buried Intake Pipes and Forebay is provided in revised CCNPP Unit 3 FSAR Sections 3.8.4, 3.8.5 and Appendix 3E as shown in Enclosure 3.

The potential for interaction of Non-seismic Category I structures with Seismic Category I structures, including applicable seismic design criteria, is addressed in updated CCNPP Unit 3 FSAR Section 3.7.2.8[1]. The analysis and design descriptions of Non-seismic Category I structures are not required in the FSAR per NRC RG 1.206 and NUREG 0800 (SRP) 3.8.4. Hence, the analysis and design descriptions of the Sheet Pile Wall, Baffle Wall, and CWS MWIS are not included in revised CCNPP Unit 3 FSAR Section 3.8.4.

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2 UN#10-062, UniStar Nuclear Energy letter from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 182, System Quality Group Classification, dated March 12, 2010.

Part 3

As depicted in CCNPP Unit 3 FSAR Figures 2.4-51 and 9.2-4, the CWS MWIS is located adjacent to the Forebay and between the two safety-related buried Intake Pipes. The CWS MWIS is situated about 100 feet north of the UHS MWIS. As described in the response to Part 2, the Forebay has been classified as Seismic Category I. Since the CWS MWIS shares a common foundation basemat with the UHS MWIS and Forebay, and has walls integrally connected with Forebay walls, the CWS MWIS was changed to Seismic Category II as part of the response to RAI 182[2]. The concrete portion of CWS MWIS is designed to the same criteria as the UHS MWIS.

The potential interaction of the CWS MWIS with the Seismic Category I structures in the vicinity (i.e., Forebay, Intake Pipes and UHS MWIS) is addressed in the response to Part 2.

Part 4

The Nuclear Auxiliary Building (NAB) is considered a Radwaste Seismic structure (designed and analyzed to meet the commitments for RW-IIa structures in Regulatory Guide (RG) 1.143), and is classified as a Seismic Category II structure due to its close proximity to the Nuclear Island. The Radioactive Waste Processing Building (RWPB) is considered a Radwaste Seismic structure and is designed and analyzed to meet the commitments for RW-IIa structures in RG 1.143.

RG 1.206 Section C.I.3.8.4 addresses the design of Seismic Category I structures other than containment. The NAB and RWPB are not Seismic Category I structures and are not safety-related structures. Therefore, a description of the analysis and design results for these structures is not required to be addressed other than for interaction with Seismic Category I structures. Interaction of non-Seismic Category I structures with Seismic Category I structures is addressed in Section 3.7.2.8 of the U.S. EPR FSAR. General arrangement drawings and descriptions of these structures are provided in Section 1.2 of the U.S. EPR FSAR. CCNPP Unit 3 did not depart from the U.S. EPR FSAR for these structures.

COLA Impact:

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 144

Question 03.08.04-2

Calvert Cliffs Unit 3 FSAR Section 3.8.4.3.1 describes the design loads used for Seismic Category I structures other than containment and containment internal structures. Provide additional information to address the items listed below.

1. Section 3.8.4.3.1 identifies the Severe Environmental Loads for the Standard Project Hurricane (SPH) and Extreme Environmental Loads for the Probable Maximum Hurricane (PMH). Provide the location in the FSAR where all of the specific quantitative data for these loads are developed. Describe how the hurricane parameters given in this section are used to calculate the pressures to be applied to the structures. Since the information provided in Section 3.8.4.3.1 only appears to be fluid pressure loads, explain what quantitative wind load is used in conjunction with the SPH and PMH for the site-specific structures and identify where this information is presented in the FSAR. Also, explain what wind loading identified as W is used for the other load combinations included in U.S. EPR FSAR Section 3.8.4.3.2 that do not include PMH and SPH.

2. Section 3.8.4.3.1 states that "the UHS Makeup Water Intake Structure (MWIS) and UHS Electrical Building are designed to withstand a peak positive overpressure (due to postulated explosions) of at least 1 psi without loss of function." Provide the basis for selection of this quantitative overpressure loading and explain how this criterion is used to demonstrate that an explosion on transportation routes (e.g., railway, highway, or navigable waterway) is not likely to have an adverse effect on plant operation or to prevent a safe shutdown of the plant. Confirm whether the evaluation for explosions is performed in accordance with NRC Regulatory Guide 1.91, Rev. 1, "Evaluations of Explosions Postulated to Occur on Transportation Routes near Nuclear Power Plants."

3. For the site-specific structures, some information is provided for hurricane loads and pressure loads due to explosions. For the site-specific structures provide a description of all the other applicable loads or explain whether the identical description and quantitative data presented in the EPR FSAR are utilized for the CCNPP Unit 3 structures as well.

Response

Part 1

The standard project hurricane (SPH) is not developed in the FSAR. The SPH is a smaller storm than the probable maximum hurricane (PMH) that is defined and discussed in CCNPP Unit 3 FSAR Section 2.4.5. The SPH assumes a surge elevation of 12.1 ft NGVD 29 (including antecedent water levels), a 10-minute average 10-m high sustained wind speed of 76 mph, a spectral significant wave height of 7.8 ft, a spectral peak wave period of 4.5 sec, a limiting shallow water wave period of 12.8 sec, and a 0.15% exceedance Rayleigh distributed (design) wave height of 14.1 ft.

The wave pressure distributions on the UHS MWIS and UHS Electrical Building exterior walls, and MWIS roof, which are exposed to direct wave actions, are obtained using the methodology available in the Coastal Engineering Manual (CEM)[3]. As described in the CEM, the total wave pressure on a structure is the combination of hydrostatic and hydrodynamic wave pressures. The hydrostatic pressure distributions are estimated based on storm surge still water levels at the structures corresponding to the SPH or PMH event.

The hydrostatic pressure distribution is combined with the hydrodynamic pressure distribution from the design waves. The Goda formula is used to estimate the hydrodynamic wave pressure on vertical and inclined exterior surfaces assuming that the design wave breaks on the structure. Based on the recommendation in the CEM, the design wave height for the shorefront exterior surfaces (east and north walls) is selected to be the 0.15% exceedance Rayleigh distributed wave height (or the average wave height of the highest 1/250 waves in a wave train). The wave pressure distributions on other exterior surfaces (west and south walls) of the UHS MWIS and UHS Electrical Building exposed to wave actions are estimated based on corresponding applicable breaking wave heights.

The dynamic wave pressures on the UHS MWIS open deck, UHS Electrical Building roof, and Forebay walls, which would be submerged and not be subject to direct wave impact during both the SPH and PMH maximum surge conditions are obtained using the stream function wave theory[4].

The following wind speeds (based on 3-sec wind gust at 10-m high) are determined to act concurrently with the hurricane waves and are presented in FSAR Section 3.8.4.3.1:

- SPH wind speed: 110 mph
- PMH wind speed: 195 mph

However, as discussed in U.S. EPR FSAR Sections 3.3.1 and 3.3.2, the U.S. EPR standard design is based on the normal (severe environment) and tornado (extreme environment) wind speeds of 145 mph and 230 mph, respectively. The site-specific SPH and PMH wind speeds are enveloped by these U.S. EPR standard wind design parameters. To be conservative, the concurrent hurricane wind pressure for design of site-specific Seismic Category I structures in the intake area is based on the U.S. EPR standard design wind speeds of 145 and 230 mph, for SPH and PMH, respectively, utilizing the procedures presented in Chapter 6 of ASCE 7-05[5].

For the Seismic Category I structures in the CCNPP Unit 3 powerblock area, the SPH and PMH wind loads are not explicitly considered because these are enveloped by the standard wind and tornado loads, respectively, as discussed above. Structures in the powerblock area are not affected by the hurricane fluid loads due to much higher grade elevation.

The wind loading identified as W in the other load combinations in Section 3.8.4.3.2 of the U.S. EPR FSAR are in accordance with ASCE/SEI Standard 7-05 as explained in the U.S. EPR FSAR, Tier 2, Section 3.3.1. It does not include PMH and SPH.

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3 Coastal Engineering Manual. Engineering Manual EM 1110-2-1100, U.S. Army Corps of Engineers, 2006.

4 Evaluation and Development of Water Wave Theories for Engineering Application. Special Report No. I. Coastal Engineering Research Center, U.S. Army Corps of Engineers, November 1974.

5 Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-05, American Society of Civil Engineers, 2005.

Part 2

The safe (allowable) distance of each Seismic Category I structure from various transportation routes for hazardous chemicals or from chemical storage locations on site was determined in accordance with NRC Regulatory Guide (RG) 1.91, Revision 1, as described in CCNPP Unit 3 FSAR Section 2.2.3.1.1. The evaluation for explosions was performed using TNT equivalency concepts described in RG 1.91 and NUREG-1805. RG 1.91 cites "1 psi" as a conservative value of peak positive incident overpressure. Below 1 psi, no significant structural damage would be expected to impact plant operation or to prevent safe shutdown of the plant. The results documented in FSAR Tables 2.2-8 and 2.2-9 show that for the applicable distances to explosions, the maximum peak overpressures developed are less than 1 psi for the CCNPP Unit 3 safety-related structures. As such, overpressure loading (due to a postulated explosion) on either the UHS Makeup Water Intake Structure or the UHS Electrical Building is not likely to have an adverse effect on plant operation or to prevent a safe shutdown of the plant. Therefore, 1 psi peak positive incident overpressure is conservatively considered in the design of these structures, as described in CCNPP Unit 3 FSAR Section 3.8.4.3.1.

Part 3

U.S. EPR FSAR Section 3.8.4.3.1 describes design loads on the other Seismic Category I structures which are also applicable for the analysis and design of site-specific Seismic Category I structures, with the following exceptions, as reflected in revised CCNPP Unit 3 FSAR Section 3.8.4.3.1:

<u>Normal Loads</u>

- Live loads (L) — The design live load due to rain, snow and ice is based on the normal and extreme winter precipitation events described in CCNPP Unit 3 FSAR Section 2.3.1.2.2.12.
- Thermal loads (T_o) — Effect of thermal loads are negligible based on site-specific temperature parameters and the guidelines provided in ACI 349.1R-07 ("Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures.")
- Soil loads and lateral earth pressure (H) — Soil loads and lateral soil pressure are calculated based on site-specific soil parameters and groundwater table. Unit weights for the structural backfill used for design are as follows:
    - Moist unit weight: 149 pcf
    - Saturated unit weight: 153 pcf

  Lateral earth pressure coefficients are defined in updated CCNPP Unit 3 FSAR Table 2.5-58[6]. An at-rest earth pressure coefficient of 0.5 is used conservatively for the design, the groundwater table in the intake area is at about Elevation 3 ft. Additionally, a minimum normal surcharge load of 500 psf and compaction-induced surcharge loads are considered in the calculation of the total lateral earth pressure.

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6 UN#09-427, UniStar Nuclear Energy Letter from Greg Gibson to Document Control Desk, U.S. NRC, Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5, dated October 9, 2009.

Severe Environmental Loads

- Operating basis earthquake (OBE) — The OBE is defined in updated CCNPP Unit 3 FSAR Section 3.7.1[1], and is essentially one-third of the site-specific safe shutdown earthquake (SSE). As such, OBE loads are not explicitly considered for the design of site-specific Seismic Category I structures.

Extreme Environmental Loads

- Safe shutdown earthquake (E') —
    - The loads associated with the SSE are based on the CCNPP Unit 3 site SSE design spectrum. The site-specific SSE, defined in updated CCNPP Unit 3 FSAR Section 3.7.1[1], has a peak ground acceleration of 0.15 g.
    - Dynamic soil pressure is calculated based on site-specific soil parameters and groundwater elevation defined in updated CCNPP Unit 3 FSAR Section 2.5.4. Effects of dynamic soil pressure on the intake structures are captured automatically in the soil-structure-interaction (SSI) analysis described in updated Section 3.7.2[1].
- External flood loads — External flood loads are governed by PMH flood level.

Abnormal Loads

Since there is no postulated high-energy pipe break accident for site-specific Seismic Category I structures, abnormal loads are not applicable.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 229

Question 03.08.04-3

Calvert Cliffs Unit 3 FSAR Sections 3.8.4.3.1, 3.8.5.5.2, and 3.8.5.5.3 identify that the EPR certified design groundwater level is exceeded in 2 instances, based on site-specific groundwater analyses. From information provided in the License Renewal application for Units 1 and 2, the staff is aware that there is an underground drain system for Units 1 and 2, whose purpose is to maintain the groundwater at a level lower than would naturally occur. The staff requests the applicant to provide the following information for Unit 3:

1. Will this existing drain system be relied on to maintain the Unit 3 groundwater at a level lower than would naturally occur? If so, describe quantitatively the estimated effect on the level of the groundwater; describe the operating experience and current condition of the drain system; describe any repairs/upgrades that will be implemented; and describe the maintenance program that will be relied on to ensure continued functioning of the existing drain system throughout the Unit 3 operating life.

2. Will a new underground drain system be installed for Unit 3, to maintain the Unit 3 groundwater at a level lower than would naturally occur? If so, describe quantitatively the estimated effect on the level of the groundwater; and describe the maintenance program that will be relied on to ensure continued functioning of the new drain system throughout the Unit 3 operating life.

3. If either existing or new underground drain system(s) are relied upon, then explain why the system(s) are not identified as safety related systems.

Response

Part 1

The existing underground drain system for CCNPP Units 1 and 2 will not be relied on to maintain the Unit 3 groundwater level.

Part 2

Based on the most recent modeling of post-construction groundwater as described in the response to RAI 101[7], the water table in the powerblock area will be approximately 30 ft below the finished grade. This water table level is well below the specified U.S. EPR FSAR groundwater level of 3.3 ft below finished grade.

Therefore, a system to maintain the CCNPP Unit 3 groundwater level lower than would occur naturally post-construction is not required.

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[7] UN#10-122, UniStar Nuclear Energy letter from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 101, Groundwater, dated May 3, 2010.

Part 3

Neither an existing nor a new underground drain system will be relied on for operation of CCNPP Unit 3.

COLA Impact

No changes to the CCNPP Unit 3 COLA are required as a result of this response.

RAI 144

Question 03.08.04-4

Calvert Cliffs Unit 3 FSAR Sections 3.8.4.6.1 and 3.8.5.6.1 state that "all natural soils at the site are considered aggressive to concrete." Based on this statement, address the following items:

1. Describe in detail, the waterproofing system that is used for all below grade concrete structures including the buried electrical duct banks and buried piping. The description should include the type of waterproofing membrane, material composition, thickness, type of joints for the membrane, and installation process. For the installation process, explain how it is assured that the waterproofing membrane will not be damaged in any manner.

2. Sections 3.8.4.6.1 and 3.8.5.6.1 indicate that the waterproofing system in combination with improved concrete mix design will adequately protect the below-grade foundations (walls and basemats) and buried duct banks. Reference is also made to ACI 201.2R-01 (Guide to Durable Concrete) and ACI 515.1R-79 (Guide to the Use of Waterproofing, Damp Proofing, Protective, and Decorative Barrier Systems for Concrete) (ACI, 1985). Provide more details on the specific measures that are being specified to ensure that no degradation of the concrete foundations and buried duct banks will occur over the potential 60 year design life of the plant. This should include a quantitative discussion of the aggressiveness of the soil/groundwater, the specific concrete mix design to be specified, which recommendations of ACI 201.2R and ACI 515.1R will be specified, and the construction procedures that will be followed to ensure durable and dense concrete. Will rubber water stops be utilized at all construction joints that may occur up to grade elevation? Additional questions related to the use of improved concrete mix design are contained in RAI 3.8-11(Internal).

3. Describe the operating experience for other below grade reinforced concrete structures that currently exist at the site which contain similar waterproofing membranes and are also exposed to comparable aggressive groundwater over long periods of time.

4. Provide vendor test data or other operating experience which demonstrates that the type of waterproofing membrane to be used has adequate water-retarding properties under aggressive saturated soil conditions for long periods of time without degrading.

Response

Part 1

Non-aggressive structural fill from off-site borrow sources will support and surround concrete structures, duct banks, and buried pipes, thereby preventing prolonged direct contact of buried concrete with any aggressive native soils. Thus, the waterproofing system described in this response functions to minimize water intrusion and eliminate direct contact of aggressive groundwater with the buried concrete, where applicable.

A waterproofing system will be used to protect structural foundations and buried walls exposed to post-construction aggressive groundwater below approximately El. 55 ft. in the powerblock area. The waterproofing system for nuclear island common basemat will consist of two layers.

The inner layer is a liquid-applied or high density polyethylene (HDPE) sheet waterproofing membrane layer will be applied to the embedded concrete walls from about one foot above the finished grade to the bottom of the wall. A vertical drainage geonet layer will then be attached to the wall. The second layer, a HDPE geomembrane will cover the drainage geonet and will form the primary waterproofing membrane layer. This geomembrane is attached to the below-grade concrete walls at a level about two feet above the highest projected post-development groundwater level and will extend over the entire immersed surfaces of the foundation walls. A thin sand layer will be placed outside the vertical HDPE prior to completing the burial of the foundation walls with structural fill.

For the protection of the concrete foundations, a sheet of textured HDPE sandwiched within two sand layers will be installed beneath the mudmat of the structure and will be contiguous with the primary HDPE geomembrane on the buried walls. A system to monitor and pump out any water collected within the resulting geomembrane envelope will also be installed. Additional discussion on the monitoring system is provided in response to Question 03.08.04-12. The waterproofing system concept is shown in revised CCNPP Unit 3 FSAR Figure 3.8-6 provided in Enclosure 3.

In the intake area, a waterproofing membrane will envelope the buried walls and foundations of the UHS Electrical Building to prevent entry of water into the facility. The membrane will be extended about one foot above the finished grade. Since the groundwater in the intake area is not aggressive, a monitoring system will not be installed.

Buried concrete duct banks and pipes will not be exposed to low pH groundwater at most locations on the site. Where buried concrete duct banks may be occasionally exposed to low pH groundwater, liquid-applied or geomembrane waterproofing will be applied. No monitoring system will be installed for the buried concrete duct banks. The protective coating for buried piping is described in response to Question 03.08.04-12, Part 3.

Where used, liquid-applied waterproofing will consist of acid-resistant coatings, such as bituminous, coal tar, epoxy, polymer-rubber, urea and/or urethane components, applied in one or more coats to achieve a thickness recommended by the respective manufacturer. The waterproofing will accommodate concrete shrinkage, thermal shrinkage, thermal cracking and expansion, and other minor movements.

The vertical drainage layer will consist of a thin diamond shaped, extruded HDPE geonet sheet, with high in-plane transmissivity and compressive strength, which will be attached to the wall between the secondary and the primary membranes. The geonet will transport any leaking water to the bottom of the foundation.

The primary geomembrane will be made of HDPE with a nominal thickness of 60-80 mils, and will be installed with fully-welded seams. It is regularly used with accessories, e.g., embedments, boots for penetrations, detailed connection fittings and with geonet. Material specified will minimize any possibilities of environmental stress cracking. The tops of vertical sheets will be attached to the wall using standard embeds.

The sand to be placed adjacent to the HDPE geomembrane will be ASTM C33 sand, or similar.

Materials, accessories, batch qualifications, film thickness, cure times, application and installation techniques, repair procedures, quality control/quality assurance procedures, specifications and details will be developed as part of the detailed design. Details of the geomembrane design will be verified based on the results of site specific qualification testing performed using standard test protocols and material requirements as specified by ASTM or other standards organizations.

Wall surfaces will be prepared and the secondary waterproofing will be applied and checked for film thickness and discontinuities. Secondary waterproofing will be re-inspected and, if needed, repaired prior to installation of the geonet. The geonet will then be applied, followed by installation of the primary geomembrane.

Horizontal geomembrane will be installed on a compacted sand bed and then covered with sand. Sand used will be cleared of materials which could damage geomembranes and placed so as to furnish firm support and minimize wrinkling and prevent damage to the geomembrane. Geomembranes will be protected from damage by workers, construction equipment, compaction loads, and exposure to temperature extremes and sunlight.

Work will be per the manufacturers' requirements and procedures, criteria developed by the testing program, and Quality Control/Quality Assurance (QC/QA) requirements. Requirements imposed on the waterproofing system will address manufacturing, material acceptance, installation requirements, full-time monitoring of installation, and testing, as applicable.

Part 2

As discussed in updated CCNPP Unit 3 FSAR Section 2.5.4[6], Non-aggressive structural fill obtained from off-site borrow sources will support and surround below-grade concrete structures, duct banks, and buried pipes, thereby preventing direct exposure of buried concrete to on-site aggressive soils. Therefore, the discussion in this response is limited to the aggressive nature of groundwater in the powerblock and intake area.

pH:

The pH of the groundwater in the powerblock area ranges from 4.5 to 6.9. Where the pH falls below 5.5 (threshold for aggressive groundwater per SRP 3.8.5 Acceptance Criteria II.7), buried concrete will be protected from prolonged exposure to low pH groundwater using the waterproofing system described in the response to Part 1. The pH of the groundwater in the intake area is approximately 7.4 and is not considered aggressive for concrete.

While the waterproofing system will effectively isolate the concrete from aggressive groundwater, its presence is ignored for the consideration of sulfate and chloride exposure.

Sulfate:

The sulfate concentration in the site soils has a mean value of approximately 5500 mg/kg and a maximum value of 31,300 mg/kg. However, the sulfate concentration in the groundwater is much lower with a maximum value of 365 mg/l. While the soil contains high amounts of sulfate, the groundwater does not. This clearly indicates that the sulfate in the soil is highly insoluble and immobile. As the in-situ soils will be removed and replaced with structural fill beneath and around the sides of the concrete structures and duct banks, the concrete will be proportioned to resist the sulfate present in the groundwater.

ACI 201.2R-01 Table 2.3 classifies this concentration of sulfate in the groundwater as a Class 1 exposure and dictates that a Portland cement conforming to ASTM C 150, Type II (or equivalent) and a water/cementitious (w/cm) ratio of 0.50 or less is required.

Chloride:

The maximum, measured chloride concentration in the site groundwater is 370 mg/l with an arithmetic mean of 45 mg/l.

For purposes of comparison, seawater is typically considered to have a chloride concentration of approximately 19,000 mg/l. The Environmental Protection Agency secondary standard for chlorides in drinking water is 250 mg/l.

Based on the arithmetic mean, the chloride concentration is less than 20% of the secondary standard for drinking water.

Thus, the penetration of chloride-ion for buried concrete at the site which is not in contact with brackish water is of no significance. For concrete structures that are in contact with brackish water, protection against chloride is achieved by limiting w/cm ratio as discussed in the subsequent paragraphs.

Concrete Mixtures:

Concrete will be batched, mixed, placed, consolidated, finished, and cured in accordance with the recommended practices contained in the ACI Manual of Concrete Practice.

Concrete mixtures will be proportioned to meet the strength requirements established by the design. All concrete mixtures for structural applications and duct banks for safety-related applications will have a maximum w/cm ratio of 0.45. A w/cm ratio of 0.40 will be used for structures exposed to brackish water, e.g., intake structures. About 20 to 25% of the Portland cement in all concrete mixtures will be replaced with fly ash conforming to ASTM C618 Class F to limit temperature gain thus reducing the peak hydration temperature and permeability of the concrete. Concrete will be placed in accordance with ACI 304R and vibrated in accordance with ACI 309R. Concrete will be cured as described in ACI 308R.

Waterstops:

PVC waterstops will be used.

Part 3

CCNPP Units 1 and 2 do not have a similar waterproofing system and are not exposed to the same groundwater chemistry expected to exist at CCNPP Unit 3. Therefore the operating experience at CCNPP Units 1 and 2 is not considered relevant for CCNPP Unit 3.

Part 4

HDPE geomembrane and geonet are widely and successfully used products for underground vertical and horizontal waterproofing barriers, and as waste containing barriers. The geomembrane is suitable for site conditions, water resistant at the site post-construction hydrostatic heads, not damaged by low pH water and other chemicals to which it may be exposed, and suitable for overlying structural loads. Many studies by the industry and the Environmental Protection Agency (EPA) have shown HDPE to be resistant to a wide variety of chemicals and to be highly durable. [8] [9] [10]

Furthermore, laboratory simulation and testing by the Geosynthetic Research Institute indicates that the HDPE geomembrane has an expected useful life many times that of the 60 years design life of the power plant. [11]

Construction and QA/QC procedures will ensure that the geomembrane is not damaged or subject to prolonged exposed to ultraviolet (UV) and that seams and seals are made properly. However, some leakage through the geomembrane may occur. This leakage will be conveyed by the vertical geonet and/or sand above the horizontal membrane to the drain sumps from where it will be pumped out.

Liquid applied or HDPE sheet waterproofing, applied to the walls as secondary waterproofing beneath the post-development groundwater level and as damp-proofing above the post-development groundwater level, are also standard, widely durable used products with successful performance under comparable conditions. The geonet applied between the primary geomembrane and the secondary waterproofing will convey any leakage away from the concrete surface to the monitoring drain located below the foundations, thereby preventing prolonged exposure of the waterproofed concrete walls to groundwater.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

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8 Assessment of Barrier Containment Technologies: A Comprehensive Treatment for Environmental Remediation Applications, S.A. O'Donnell, R.R. Rumer, and J.K. Mitchell, eds. Sponsored by US DOE, US EPA, and DuPont Company. NTIS PB96-18053, 438 pp. 1995 (Chapter 5, Pg. 99, 109, and 111).

9 Carlowitz: Kunststofftabellen-3. Auflage, Chemicals Resistance Table – Low Density and High Density Polyethylene, ISO/TR 7472.7474, ART 254 10.12.1999 Ed. 2.

10 Bellen, G., R. Corry, and M.L. Thomas, (1987). "Development of Chemical Compatibility Criteria for Assessing Flexible Membrane Liners, EPA 600/2-87/067, National Sanitation Foundation, Ann Arbor, MI., 510 pp.

11 Koerner, R.M., Y.G. Hsuan, and G.R. Koerner, (2005). Geomembrane Lifetime Prediction: Exposed and Unexposed Conditions, GRI White Paper No. 6, Geosynthetic Institute, Folsom, PA, 19 pp.

RAI 144

Question 03.08.04-5

Calvert Cliffs Unit 3 FSAR Section 3.8.4.3.2 presents two additional load combinations for the UHS MWIS and UHS Electrical Building to address the hurricane loadings SPH and PMH. The Severe Environment SPH load combination appears to correspond to one of the Service Load Combinations presented in the EPR FSAR and ACI 349, when the wind load W is replaced by the hurricane load SPH. The Extreme Environment PMH appears to correspond to one of the Factored Load Combinations that are presented in the EPR FSAR and ACI 349, when the tornado load Wt is replaced by the hurricane load PMH. Address the following items related to these load combinations:

1. Explain why these two load combinations are only applicable to the UHS MWIS and UHS Electrical Building, and not to the other Seismic Category I structures as well.

2. The load combination U = (0.75)(1.4D + 1.4F + 1.7L + 1.7H + 1.7W + 1.7To +1.7Ro) appears in the EPR FSAR and ACI 349. Explain why a load combination corresponding to U = (0.75)(1.4D + 1.4F + 1.7L + 1.7H + 1.7SPH (replacing W) + 1.7To +1.7Ro) is not considered.

3. In order to be consistent with the Factored Load Combinations in the EPR FSAR and ACI 349 that contain Wt (tornado wind), explain why the load To was omitted in the "Extreme Environment PMH" load combination presented in Section 3.8.4.3.2 of the CCNPP Unit 3 FSAR.

Response

Part 1

A hurricane event includes hurricane wind and concurrent hurricane flood. Loading effects from hurricane wind is addressed in response to Question 03.08.04-2 Part 1. The response presented herein addresses the hurricane flood event.

Structures in the powerblock area (in and around Nuclear Island) with a nominal grade elevation of 85 feet are located above the storm surge and wave run-up associated with a probable maximum hurricane (PMH) event. Therefore, PMH flood load is not applicable to the design of Seismic Category I structures in the powerblock area. Similarly, standard project hurricane (SPH) flood load is also not applicable to these structures since SPH flood level is lower than the PMH flood water level.

Seismic Category I buried electrical bank and piping in the intake area is subjected to PMH and SPH flood loads. This is clarified in revised CCNPP Unit 3 FSAR Section 3.8.4.3.2.

Part 2

There are two load combinations, which include wind load (W), in accordance with U.S. EPR FSAR Section 3.8.4.3.2:

$$U= 1.4(0 + F) + 1.7(L + H + W + R_o)$$

$$U= (0.75)(1.40 + 1.4F + 1.7L + 1.7H + 1.7W + 1.7T_o + 1.7R_o)$$

SPH load combinations are obtained by replacing W with SPH in the above load combinations. However, in the second load combination, effect of T_o is deemed to be negligible for the UHS MWIS and UHS Electrical Building per Section 1.3 of ACI 349.1 R-07 (Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures). With T_o eliminated and W replaced by SPH, the second load combination will not govern and will be enveloped by the first load combination. It is for this reason U = (0.75)(1.40 + 1.4F + 1.7L + 1.7H + 1.7SPH (replacing W) + $1.7T_o$ +$1.7R_o$) is not explicitly considered for design.

Part 3

As indicated in the response to Part 2 of this question, effect of T_o is deemed to be negligible for the UHS MWIS and UHS Electrical Building per Section 1.3 of ACI 349.1 R-07 and is thus omitted from the load combination involving PMH.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 144

Question 03.08.04-6

Calvert Cliffs Unit 3 FSAR Section 3.8.4.4.5 provides a limited description of the analysis and design procedures for buried electrical duct banks and buried Essential Service Water pipes. The first COL Item listed in Section 3.8.4.4.5 indicates that a COL applicant will describe the design and analysis procedures for the conduit and buried pipe. Section 3.8.4.4.5 refers to Section 3.7.3 for the seismic design of buried duct banks and buried pipe. Information for the analysis and design procedures for all of the other loads is lacking.

Therefore, provide a description of the analysis and design procedures for all of the other loads imposed on all the buried duct banks and buried pipes. This description should include the procedures for analysis and design under vertical earth loads, permanent surface loads, surface live loads, internal pressure (for pipe), fluid transients (if applicable), buoyancy, thermal expansion (if applicable), and frost effects (e.g., heave for pipes placed above the frost line).

This description should also clearly state

(1) whether the approach follows the analysis and design procedures presented in EPR FSAR Section 3.8.4.4.5, including the AREVA report entitled U.S. Piping Analysis and Pipe Support Design Topical Report (Reference 37 of the EPR FSAR) for buried piping and

(2) the extent to which the procedures in EPR FSAR Section 3.8.4.4.5 and EPR Reference 37 are used for buried electrical duct banks.

If a different approach is used for either buried duct banks or buried pipe, provide a detailed description of the approach used.

Since the groundwater table is probably above the buried electrical duct banks, explain what types of joints are used and what provisions are made to prevent water intrusion.

Response

The analysis and design procedures used for buried pipes, for the imposed loads, are the same as described in Section 3.8.4.4.5 of U.S. EPR FSAR and Section 3.10 of ANP-10264NP-A[12].

The analysis and design procedures for buried electrical duct banks for various loadings are in accordance with U.S. EPR FSAR Section 3.8.4.4.5. The loads are described in the Section 3.10 of ANP-10264NP-A[12]. However, the structural acceptance criteria for the buried electrical duct banks are in addressed in revised CCNPP Unit 3 FSAR Section 3.8.4.5.

In areas where the buried electrical duct banks will be below groundwater table, the buried

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[12] U. S. EPR Piping Analysis and Pipe Support Design, Revision 0, AREVA NP Inc.,Topical Report ANP-10264NP-A, November 2008.

electrical duct banks will have water-tight construction joints utilizing water stops. The joints between buried duct banks and manholes will have PVC water stops to prevent water intrusion.

Buried electrical duct banks will have drain pipes at the bottom, and will be constructed such that they slope from manhole to manhole. The low point manhole will have a sump with a pump for collecting and disposing water that could penetrate the buried duct banks and manholes.

The post-construction groundwater table in the powerblock area is about 30 feet below grade. Where necessary to prevent the direct prolonged exposure of duct banks concrete from the low pH groundwater, waterproofing, as described in response to Question 03.08.04-04, will be used.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 144

Question 03.08.04-7

Calvert Cliffs Unit 3 FSAR Sections 3.8.4.4.6 (Other Seismic Category I Structures – Design Report) and 3.8.5.4.5 (Foundations - Design Report), state "No departures or supplements." Since there are three site-specific Seismic Category I structures defined in the FSAR, a Design Report is required for each of these structures. Therefore, provide a Design Report for the UHS Makeup Water Intake Structure, UHS Electrical Building, and the buried electrical duct banks and buried piping. The Design Reports should be prepared in accordance with the guideline described in NRC SRP 3.8.4, Appendix C. The Design Reports could be separate documents referenced by the FSAR or included as part of the FSAR as an Appendix. If Appendix 3E.4 is used for the purpose of the Design Reports, then it would need to be expanded to include the other information described in SRP 3.8.4, Appendix C.

Response:

Design information and design results required in the Design Report per SRP 3.8.4, Appendix C, for the site-specific Seismic Category I Ultimate Heat Sink Makeup Water Intake Structure and Electrical Building are updated and expanded in revised CCNPP Unit 3 FSAR Appendix 3E.4.

The seismic classification of the Forebay has been changed to Seismic Category I. Therefore, the design information and design results required in the Design Report per SRP 3.8.4, Appendix C are also provided for the Forebay in revised CCNPP Unit 3 FSAR Appendix 3E.4.

The analysis and design of buried duct banks and pipes will be performed during detailed design engineering. The design codes and analysis methodology, to be used for buried piping and duct banks during the detailed design engineering, are provided in the responses to Question 03.08.04-6 and RAI 63, Question 03.07.03-1[13]. Design information and design results required in the Design Report per SRP 3.8.4, Appendix C, for Seismic Category I buried electrical duct banks and piping will be incorporated into revised CCNPP Unit 3 FSAR Appendix 3E.4, prior to closure of Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) described in updated Table 2.4-9[14] in COLA Part 10, Appendix B.

COLA Impact:

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

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[13] UN#09-320 from Greg Gibson 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, and RAI No. 65, Seismic System Analysis, dated July 15, 2009.

[14] UN#10-160 from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 118, Structural and Systems Engineering Inspections,Tests, Analyses, and Acceptance Criteria, dated June 18, 2010.

RAI 144

Question 03.08.04-8

Calvert Cliffs Unit 3 FSAR Section 3.8.4.4.7 provides information about the design and analysis procedures for the UHS Makeup Water Intake Structure and the UHS Electrical Building. The discussion of design and analysis procedures for the UHS Makeup Water Intake Structure and the UHS Electrical Building needs to be expanded to provide more detailed information. The additional information should include:

1. For the UHS Makeup Water Intake Structure:

   a. For determining member forces in the structure for design purposes, provide more detailed information on the finite element model (FEM) and analysis than that described in Section 3.8.4, 3.8.5, and Appendix 3E. This should include information on: (1) soil representation used in the FEM (e.g., why pinned supports rather than soil springs), (2) how equivalent static loads are determined and then applied, (3) consideration of any local dynamic amplification for slabs and walls for seismic loading, (4) seismic load application (were loads applied simultaneously in three directions or applied separately? If separately, how are the responses combined? Due to non-symmetry conditions are seismic loads considered to act in plus and minus horizontal directions?), (5) representation of water within the structure and outside the structure.

   b. If the same model and approach described in FSAR Section 3.7.2 is used for representation of water, simply stating that it was done in accordance with ACI 350.3-06 and Army Corps of Engineers Manual EM-1110-2-6051 is not acceptable. These standards have not been previously reviewed and endorsed by the NRC and many elements of these standards are not applicable to nuclear power plants. Provide a description of how the water contained within the structure and outside the structure was considered in the model for developing member forces.

   c. Explain why the concrete shear keys below the basemat are not included in the FEM and why the sloped concrete walls on the North-West side of the UHS Makeup Water Intake Structure, shown on Figure 3E.4-2, are not also sloped in the FEM on Figure 3.8-5.

   d. Provide a description of how all the loads were determined and applied to this model. This should include soil loads from dead weight, live load, surcharge, seismic, and soil passive pressure (if relied upon for stability evaluation); water pressure within and outside the building; and the hurricane induced loadings (pressure loadings from wind, storm surge and wave run-up).

   e. Section 3.8.4.4.7 states that the "results from the GT STRUDL static analysis are used to design reinforced concrete shear walls and slabs according to provisions of ACI 349-01 (ACI, 2001a) (with supplemental guidance of Regulatory Guide 1.142 (NRC, 2001)), ACI 350-06 (ACI, 2006a) and ACI 350.3-06 (ACI, 2006b)." These ACI standards have not been previously reviewed and generically endorsed by the NRC and some elements of these standards are not applicable to nuclear power plants. Furthermore, the referenced Regulatory Guide 1.142 endorses ACI 349-97, not ACI 349-01. Therefore, specifically identify which sections/provisions in the three ACI referenced standards are used for design and describe how they compare to ACI 349-97, supplemented by Regulatory Guide 1.142. Note that this item, related to the appropriate ACI Standard(s), is also applicable to the UHS Electrical Building and to the buried electrical conduit duct banks.

2. For the UHS Electrical Building:

   Section 3.8.4.4.7 states "Due to its relative simplicity and treatment as a soil inclusion, the design of the embedded UHS Electrical Building is performed by manual calculations. Reinforced concrete shear walls and slabs are designed in accordance with ACI 349-01 (ACI, 2001a) (with supplemental guidance of Regulatory Guide 1.142 (NRC, 2001)), ACI 350-06 (ACI, 2006a) and ACI 350.3-06 (ACI, 2006b)."

   a. Explain what is meant by the phrase "soil inclusion."

   b. Since Section 3.8.4.4 is supposed to present the design and analysis procedures, provide a description of how the manual calculations were performed for the various loads.

   c. Address the same question raised under Item 1.e above, regarding the use of the three ACI standards, as it applies to the UHS Electrical Building.

Response

As described in updated CCNPP Unit 3 FSAR Sections 3.7.1 and 3.7.2[1], the Seismic Category I Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS) is a part of a larger integrated system of structures that includes Seismic Category I Forebay and Seismic Category II Circulating Water System (CWS) Makeup Water Intake Structure. These structures are referred to as Common Basemat Intake Structures (CBIS). Separate finite element models (FEM) were developed for the CBIS and the UHS Electrical Building using GT STRUDL. The two finite element models are translated into a single soil-structure interaction (SSI) model using Bechtel computer code SASSI 2000 to perform SSI and structure-soil-structure interaction (SSSI) analyses. The responses to various parts of this question are prepared based on the new configuration of the structures.

Part 1

a) The detailed information about the integrated FEM for the new configuration is provided in updated CCNPP Unit 3 FSAR Section 3.7.2[1]. In addition, CCNPP Unit 3 FSAR Sections 3.8.4, 3.8.5 and Appendix 3E are revised to include information on the representation of soil springs for static analysis, combination of seismic and non-seismic responses and design of critical sections. Additional information on each item of this part of the question is presented in the following paragraphs.

   (1) The current static analysis of the integrated FEM utilizes soil springs based on the subgrade modulus developed using soil parameters for the intake area provided in updated CCNPP Unit 3 FSAR Section 2.5.4[8] and the structure foundation geometry and stiffness.

(2) SSI analysis generates peak element forces and moments (plate membrane and bending resultants) due to three orthogonal components of the safe shutdown earthquake (SSE) applied independently. These element forces and moments are combined using 100-40-40 rule and plus-minus directions for co-directional responses. The seismic responses are then combined with responses due to static load effects in accordance with the various applicable load combinations per ACI 349-01 (with supplemental guidance of Regulatory Guide 1.142). Since element stress resultants are extracted directly from SSI analysis results, equivalent static seismic loads are not required in the analysis and design of the intake structures.

(3) Since the slabs and walls are modeled in SASSI 2000 using a mesh of thick shell elements sufficiently refined to capture plate bending, membrane and shear deformations, the results obtained from SSI analysis automatically include the dynamic amplification effects due to local flexibility of all slabs and walls.

(4) Application of seismic loads is discussed in response to Part 1(a)(2).

(5) Hydrodynamic loads are discussed in response to Part 1(b). The hydrostatic loads within and outside the structure are discussed in the response to Part 1(d).

b) The discussion of hydrodynamic loads due to water inside the structure is provided in updated CCNPP Unit 3 FSAR Section 3.7.2[1]. Additional discussion on the subject is provided in response to RAI 159, Question 03.07.02-30[1].

c) Shear keys are not utilized in the current configuration of the structure and hence are not modeled in the FEM. The sloped wall in the FEM is the North wall of UHS MWIS. The wall is sloped at an angle of 10 degrees to the vertical. However, for simplification of the finite element model, the wall is modeled as vertical and in line with center line of Forebay wall. As described in updated CCNPP Unit 3 FSAR Section 3.7.2.3.2[1], this simplification has an insignificant effect on the global mass and stiffness distribution, and is conservative for the local response of structural panels. This simplification has little effect on the design of the sloped wall and critical sections described in revised CCNPP Unit 3 FSAR Appendix 3E.4. Localized impact on the design of operating deck slab will be addressed during the detailed design engineering.

d) U.S. EPR FSAR Section 3.8.4.3.1 describes the design loads on the other Seismic Category I structures which are also applicable for the analysis and design of the site-specific Seismic Category I structures with additions/modifications described in FSAR Section 3.8.4.3.1 and the response to Question 03.08.04-2 Part 3. The information on the requested loads is further expanded in this response.

Soil loads from dead weight: Lateral soil pressures at any depth are determined from the cumulative weight of the soil layers above, using the soil parameters provided in updated CCNPP Unit 3 FSAR Sections 2.5.4[6]. Lateral pressures are evaluated as a triangular pressure distribution but applied as equivalent uniform pressure loads on the individual elements of buried portions of the walls.

Live load: Live loads are determined as described in US EPR FSAR Section 3.8.4.3.1 and as supplemented by response to Question 03.08.04-2 Part 3. Live loads are applied as uniformly distributed pressure loads on the applicable structural elements. Live loads on the removable roof slabs are converted into equivalent concentrated loads on nodes around the openings.

Surcharge load: A nominal surcharge load of 500 psf or the actual surcharge, whichever is larger, is considered. An appropriate coefficient of lateral pressure from updated CCNPP Unit 3 FSAR Section 2.5.4[6] is considered to calculate a uniform lateral pressure on the embedded portions of the intake structures. This is applied as uniformly distributed pressure load on applicable finite elements of the model.

Seismic loads: Seismic loads are determined from SSI analysis using Bechtel computer code SASSI 2000. Additional information is provided in response to Part 1(a)(2) above.

Soil passive pressure: Passive pressure is not utilized for sliding stability evaluation of UHS MWIS.

Water pressure within and outside the building: Hydrodynamic loads are discussed in response to Part 1(b) of this question. Hydrostatic pressures from the water inside the structure are applied as uniformly distributed vertical load on the basemat and lateral pressure on both exterior and interior walls is evaluated as linearly distributed load but applied as equivalent uniform pressure on the individual applicable finite elements. Lateral hydrostatic pressure from the water in the soil outside the structure is evaluated as a triangular pressure distribution but applied as an equivalent uniform pressure load on the individual applicable finite elements. Buoyant forces are also considered and applied in upward direction as uniformly distributed loads on finite elements of the basemat.

Hurricane induced loadings (pressure loadings from wind, storm surge and wave run-up): Hurricane induced loadings are described in revised CCNPP Unit 3 FSAR Section 3.8.4.3.1. Determination of associated wave pressure loadings is described in response to Question 03.08.04-2 Part 1. These loads are evaluated and applied as linearly distributed pressures on the individual applicable elements.

e) ACI Codes:

ACI 349-01: This code, in conjunction with RG 1.142, is used to analyze and design the CBIS. Numerous sections/provisions are used for design. Detailed comparison of ACI 349-01 with ACI 349-97 is provided by AREVA response to US EPR FSAR RAI 155, Question 03.08.03-3[15].

ACI 350-06: This code is not used to design the CBIS. Reference to this code has been deleted from revised CCNPP Unit 3 FSAR Sections 3.8.4 and 3.8.5 and Appendix 3E.4

ACI 350.3-06: Sections 9.2.1 and 9.2.2 of ACI 350.3-06 are used to calculate earthquake induced impulsive and convective water loads on the CBIS. Note that ACI 350.3, which is referenced by ASCE 7-05, is a well maintained standard that reflects the state-of-art in earthquake design of concrete liquid-containing structures. This standard essentially utilizes the same equations/concepts for impulsive and convective water loads that are described in

[15] AREVA Email from Ronda Pederson to Getachew Tesfaye, Response to U.S. EPR Design Certification Application RAI No. 155, Supplement 3, dated May 29, 2009.

TID-7024[16] as the "Housner Method" and identified in NUREG-0800, SRP 3.7.3.

Applicability of ACI codes to UHS Electrical Building: Applicability is discussed with the response Part 2(c).

Applicability of ACI codes to Buried Electrical Duct Banks: ACI 349-01 (with supplemental guidance of Regulatory Guide 1.142) is used for design of buried electrical duct banks. ACI 350-06 and 350.3-06 are not used for the design of buried electrical duct banks.

Part 2

a) For SSI analysis, the UHS Electrical Building is modeled using finite elements along with the common basemat intake structures. Therefore, the phrase "soil inclusion" has been removed from the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E.4.

b) As stated in response to Part 1, the UHS Electrical Building along with the soil between the UHS MWIS and the UHS Electrical Building is part of the SASSI 2000 model used for SSI analysis. The results of SSI analysis are presented in updated CCNPP Unit 3 FSAR Section 3.7.2[1]. The static analysis is performed using GT STRUDL finite element model. The analysis and design procedures including the results are presented in revised CCNPP Unit 3 FSAR Sections 3.8.4, 3.8.5 and Appendix 3E.4. No manual calculations are employed for UHS Electrical Building analysis.

c) ACI Codes:

   ACI 349-01: This code, in conjunction with RG 1.142, is used to analyze and design the CBIS. Numerous sections/provisions are used for design. Detailed comparison of ACI 349-01 with ACI 349-97 is provided by AREVA response to US EPR FSAR RAI 155, Question 03.08.03-3[15].

   ACI 350-06: This code is not used to design the UHS Electrical Building. Reference to this code has been deleted from revised CCNPP Unit 3 FSAR Sections 3.8.4 and 3.8.5 and Appendix 3E.4.

   ACI 350.3-06: This code is not used to design the UHS Electrical Building. Reference to this code has been deleted from revised CCNPP Unit 3 FSAR Sections 3.8.4 and 3.8.5 and Appendix 3E.4.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

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[16] TID-7024, "Nuclear Reactors and Earthquakes", Division of Reactor Development, U.S. Atomic Energy Commission, August 1963.

RAI 144

Question 03.08.04-9

Calvert Cliffs Unit 3 FSAR Section 3.8.4.5 indicates that Section 3E.4 of Appendix 3E provides the details for the design of the basemat and typical wall for the UHS Makeup Water Intake Structure and the UHS Electrical Building. What is the technical basis for only selecting a typical wall for each structure? Explain why other concrete walls and slabs were not considered. Since the buried electrical duct banks and buried piping are also site-specific Seismic Category I structures, provide corresponding analysis and design information for critical sections of electrical duct banks and buried piping to represent this group of structures/components.

Response

Analysis of the UHS Makeup Water Intake Structure identified the embedded and above-grade side walls as critical elements because of their location, support and loading conditions. Unlike the embedded back wall, the embedded side walls are not supported by multiple interior walls, as shown in revised CCNPP Unit 3 FSAR Figure 3E.4-1. In addition, the side walls are subjected to more severe loading demands compared to any interior wall. Design of the side walls is presented in revised CCNPP Unit 3 FSAR Section 3E.4.4 and the reinforcement arrangement is presented in Figure 3E.4-4.

Analysis of the UHS Electrical Building identified the left side exterior wall (74 ft long wall adjacent to stairwell, see revised CCNPP Unit 3 FSAR Figure 3E.4-2) as a critical element because of its location, dimensions, support conditions, and load condition. Loads acting normal to this wall will be transferred to the roof and basemat through "one-way" action, while loads acting normal to other exterior walls will be transferred through "two-way" action. Design of the left side wall is presented in revised CCNPP Unit 3 FSAR Section 3E.4.4 and the reinforcement arrangement is presented in Figure 3E.4-6.

As stated in response to Question 03.08.04-7, the analysis and design of buried electrical duct banks and buried piping will be performed during detailed design engineering. The analysis and design methodology for buried electrical duct banks and buried piping are provided in the responses to Question 03.08.04-6 and RAI 63, Question 03.07.03-1[13]. The design report according to SRP 3.8.4, Appendix C, including the results of the analysis and design of buried electrical duct banks and buried piping will be available prior to closure of Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) described in updated Table 2.4-9[14] in COLA Part 10, Appendix B.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 144

Question 03.08.04-10

Calvert Cliffs Unit 3 FSAR Sections 3.8.4.6.1 and 3.8.5.6.1 refer to "the use of dense concrete with a low water cement ratio and improved concrete mixture design." According to Section 3.8.5.6.1, the compressive strength of the concrete for the foundation of the UHS Makeup Water Intake Structure and UHS Electrical Building is f'c = 5,000 psi. Provide information to address the following related items:

1. FSAR Sections 3.8.1 through 3.8.5 should identify any specific water cement ratios needed for the concrete mix for all Seismic Category I structures. The tables in Part 10, Section 2.4 (ITAAC) specify that the acceptance criterion is a maximum water cement ratio of 0.45 for all below grade concrete sections. Explain how this value was selected considering that usually a lower value of the water cement ratio, high compressive strength f'c, and large concrete cover over steel reinforcement are recommended for aggressive concrete surface conditions. As an example, ACI 350-01 recommends a water cement ratio of 0.40 and f'c = 5,000 psi for severe aggressive conditions. Also, clarify where in the FSAR the water cement ratios for Seismic Category II and II-SSE structures are specified.

2. In view of the aggressiveness of the soil conditions at CCNPP Unit 3, explain why the concrete compressive strength for most of the other Seismic Category I structures is less than 5,000 psi, which is the value used for the UHS Makeup Water Intake Structure and UHS Electrical Building. EPR FSAR Sections 3.8.4.6.1 and 3.8.5.6.1 indicate that 4,000 psi is specified for the foundations of Seismic Category I structures including the buried electrical duct banks. Also clarify where in the FSAR the compressive strength for the Seismic Category II and II-SSE structures is specified and address this issue for these structures as well.

Response

Part 1

Design of Seismic Category I concrete structures is governed by ASME BPVC Section III Division 2 (supplemented by RG 1.136) or ACI 349 (supplemented by RG 1.142). Selected water-cement ratios will be in accordance with the requirements of these design codes for special exposure conditions, such as concrete exposed to brackish water, chloride, and sulfate. The water cement ratios for various Seismic Category I concrete structures are presented in Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) Tables 2.4-2, 2.4-3, 2.4-6, 2.4-7, 2.4-8, 2.4-9, and 2.4-33 (This is a new table added for the Forebay by the response to RAI 118, Question14.03.02-21)[14].

According to ACI 349 Table 4.2.2, for corrosion protection of reinforcement in concrete exposed to brackish water, a maximum water-cement ratio of 0.4 should be used. Accordingly, the applicable ITAAC Tables will be revised for all the structures exposed to brackish water to satisfy the code requirement, as shown in the markup of COLA Part 10, Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) in Enclosure 5.

The water-cement ratios for Seismic Category II and II-SSE concrete structures are presented in ITAAC Tables 2.4-10, 2.4-11, 2.4-12, and 2.4-19. The water-cement ratios were added to Tables 2.4-11, 2.4-12, and 2.4-19 by the responses to RAI 118, Question 14.03.02-2G[17].

The water-cement ratios for Radwaste Seismic concrete structures are also presented in COLA Part 10 (ITAAC) Tables 2.4-4 and 2.4-5.

Part 2

According to the durability requirements presented in Chapter 4 of ACI 349, the minimum compressive strength shall be 5000 psi for concrete exposed to brackish water. For concrete exposed to sulfate, the minimum compressive strength shall be 4500 psi for severe or very severe exposure. A minimum compressive strength of 5000 psi satisfies both requirements and is specified for other Seismic Category I structures addressed in revised CCNPP Unit 3 FSAR Section 3.8.4, including buried concrete pipes and duct banks. This requirement is also applicable to Seismic Category II and II-SSE concrete structures. The concrete compressive strength for Seismic Category II and II-SSE structures is not presented in the CCNPP Unit 3 FSAR since the FSAR is prepared according to the standard format and content presented in RG 1.206, which has no requirement for such information.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

In addition, Tables 2.4-6, 2.4-7, 2.4-8, 2.4-9, 2.4-19, and 2.4-33 in COLA Part 10 will be revised as shown Enclosure 5.

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[17] UN#10-071, UniStar Nuclear Energy letter from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI No. 118, Structural and Systems Engineering – Inspections, tests, Analyses and Acceptance Criteria, dated March 31, 2010.

RAI 144

Question 03.08.04-11

U.S. EPR FSAR Sections 3.8.4.6.1 and 3.8.5.6.1 require a COL applicant that references the U.S. EPR design certification to evaluate the use of waterproofing membranes and epoxy coated rebar based on site-specific groundwater conditions. Describe the evaluation performed to determine whether epoxy coated rebar is needed in accordance with the referenced COL item.

Response

Calvert Cliffs Nuclear Power Plant Unit 3 site soils contain three aggressive agents that can affect the performance and durability of reinforced concrete.

1. Acidic groundwater with average pH of approximately 5.2 in the powerblock area (Note: Average pH of the groundwater in the intake area is approximately 7.4)
2. Chloride, and
3. Sulfate

These three agents can be potentially adverse to the concrete section in the following manner:

1. Exposure to acids and sulfate attack the hydrated Portland cement paste, and
2. Chloride, if able to penetrate the concrete and encounter the reinforcing steel, accelerates corrosion of the reinforcing steel.

After removal of in-situ soils, the structures in the CCNPP Unit 3 powerblock area and intake area will no longer be contact with in-situ soils. However, the groundwater in the powerblock area is still considered aggressive to concrete. Acidic groundwater can be very detrimental to the performance of the concrete. Concrete cannot be made resistant to long term exposure to acidic groundwater. Protection of concrete from the acidic groundwater, using waterproofing membrane system, is further discussed in the responses to Questions 03.08.04-4 and 03.08.04-12.

This protection against the acidic groundwater is neglected for purposes of evaluating the concrete for sulfate resistance and the reinforcing steel for chloride attack. Discussion of sulfate and chloride concentrations is included in the response to Question 03.08.04-4.

The effect of sulfate on hydrated Portland cement paste is well understood. To ensure service in sulfate-rich environments, a dense, impermeable concrete mixture is employed. Performance in this environment is ensured by using a concrete mixture with a relatively low water/cement ratio and an appropriate limit on the C_3A content of the Portland cement.

As concrete itself is unaffected by chloride, protection of the reinforcing steel can be provided by either an epoxy coating or a dense, impermeable concrete mixture. It was determined that epoxy coated reinforcing steel was less desirable and less reliable than use of a suitably proportioned concrete mixture. The epoxy coated reinforcing has potential for coating damage during placement of the reinforcing and the concrete. A suitable concrete mixture that will perform both functions (resistance to sulfates and impermeability to chlorides) will be developed using a supplementary cementitious material (such as fly ash) that is demonstrated by test to be resistant to sulfate as described in ACI 201.2R. This is desirable as such concrete mixtures generate less heat when curing and have improved workability.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 144

Question 03.08.04-12

Calvert Cliffs Unit 3 FSAR Sections 3.8.4.7 and 3.8.5.7 describe testing and in-service inspection requirements. The CCNPP Unit 3 below-grade concrete degradation program for aggressive groundwater/soil "provides a periodic surveillance program to monitor the condition of normally inaccessible below-grade concrete for signs of degradation." This program includes below-grade walls and buried duct banks, addressed in Sections 3.8.4.7, and foundations, addressed in Section 3.8.5.7. This in-service inspection program is limited to examination of exposed portions of below-grade concrete for signs of degradation when adjacent soil is excavated for any reason. Address the following items related to the in-service inspection requirements:

1. The staff notes that the approach to limit the in-service inspection program to examination of exposed portions of below-grade concrete for signs of degradation when adjacent soil is excavated for any reason has been used and accepted at sites where the soil is not aggressive. Therefore, provide more details about this program and why is it considered adequate for below grade concrete foundations when subjected to aggressive soil conditions. The description should include a discussion of the scope, locations, schedule, parameters inspected, inspection methods, and acceptance criteria. Also provide the technical basis for assuming that the presence of a waterproof membrane is sufficient justification to follow an in-service inspection program normally used where the soil is not aggressive.

2. Explain why the description in the FSAR refers to this as a periodic surveillance program while in a later discussion it indicates that the inspection is limited to examination of the surfaces when the adjacent soil is excavated for any reason. Provide the basis for why examination of exposed portions of below-grade concrete, when adjacent soil is excavated for any reason, is considered adequate rather than supplementing this requirement with a specified maximum time period.

3. Explain why the FSAR does not state that such a program is also applicable to buried piping considering the aggressive soil conditions present at the site.

4. For the waterproofing membrane beneath the foundation basemats and on the below grade walls, explain what type of inspection is to be specified to ensure that the waterproofing membrane has not been damaged or shows sign of degradation. Explain whether this inspection will be performed prior to the placement of soil backfill and during the periodic below-grade concrete degradation program.

5. Explain whether the monitoring and maintenance of all Seismic Category I, II, and II-SSE structures, including the site-specific structures, will be performed in accordance with the requirements of 10 CFR 50.65, supplemented with the guidance in Regulatory Guide 1.160. For the UHS Makeup Water Intake Structure and CW Intake Structure explain whether the inspections will also be performed in accordance with NRC Regulatory Guide 1.127, Rev. 1, "Inspection of Water-Control Structures Associated with Nuclear Power Plants."

Response

Part 1

As described in the response to Question 03.08.04-4, non-aggressive structural fill derived from off-site borrow sources will support and surround below-grade concrete structures, and buried concrete ductbanks and pipes, preventing direct exposure of buried concrete to on-site aggressive soils. Therefore the in-service inspection program as described in revised CCNPP Unit 3 FSAR Sections 3.8.4.7 and 3.8.5.7 is applicable for below-grade concrete to be placed above the post-construction groundwater table, or which will not otherwise be subjected to prolonged exposure to aggressive site groundwater derived from the Surficial aquifer.

Furthermore, the UHS Electrical Building, UHS Makeup Water Intake Structure, Forebay and associated buried concrete duct banks and pipes are not exposed to the low pH groundwater of the Surficial aquifer which is absent at these locations. Therefore, the in-service inspection program as described in revised CCNPP Unit 3 FSAR Sections 3.8.4.7 and 3.8.5.7 is also applicable to these structures and buried utilities. Surfaces of the below-grade concrete, when excavated for any reason, will be examined for evidence of pattern cracking, deterioration of the formed surface, aggregate exposure, paste deterioration, or degradation of the original formed surface.

Below grade concrete that will be located beneath the groundwater table in the backfilled powerblock area may be exposed to the aggressive low pH groundwater of the surficial aquifer unless alternative design provisions are incorporated. Therefore, the facility design for such structures will include a waterproofing geomembrane envelope as described in response to Question 03.08.04-4.

However, since it is possible for leakage to occur, a monitoring system (consisting of risers and drain sumps) will be installed inside the waterproofing geomembrane. The monitoring risers and drain sumps will be structure-specific and will be designed in parallel with the building foundations.

The water level within the risers and sumps will be subject to periodic monitoring to confirm that groundwater is being effectively removed and is not ponded against the concrete structure. Such monitoring will:

- occur at multiple locations in the monitoring system;
- be performed on a frequency based on the leakage rate of the primary geomembrane. Initially the monitoring frequency is expected to be high until the performance of the geomembrane is established. As the operation proceeds, the monitoring interval will be expanded, possibly to once per cycle; and
- will utilize manual techniques or electronic water level sensors.

Surrounding groundwater levels in the powerblock area will also be monitored to confirm that no other below-grade concrete is exposed to the aggressive low pH groundwater.

Since the concrete duct banks and piping will typically have shallow cover of structural fill, the condition of the buried concrete duct banks that may be exposed to low pH groundwater will be monitored by excavating the surrounding soil.

Revised CCNPP Unit 3 FSAR Sections 3.8.4.7 and 3.8.5.7 will retain the current program calling for in-service inspection of surfaces of the below-grade concrete, when excavated for any reason. These FSAR sections will also describe the monitoring system.

Part 2

Below grade concrete will be supported on, and surrounded by, non-aggressive structural fill obtained from off-site. Much of the below grade concrete will also be located where it will not be immersed in aggressive groundwater, and will therefore not suffer prolonged exposure to aggressive groundwater. For this concrete, examination of concrete surfaces will be performed when the normally inaccessible, below-grade concrete is excavated for any reason, as stated in revised CCNPP Unit 3 FSAR Sections 3.8.4.7 and 3.8.5.7, in conformance with SRP 3.8.5.

Part 3

Buried piping will be installed with protective coatings and will be embedded within non-aggressive bedding material obtained from off-site sources.

The wrapping and coatings will be selected and evaluated on the basis of industry operating experience and performance with exposure and service conditions including immersion. Selected material for use will be applied in a manner that optimizes corrosion protection and performance and maintains integrity of the applied material in a given service environment. Criteria for inspection and acceptance will be addressed in project specifications using the applicable industry standards such as AWWA C210, NACE RP0169-2002, ASTM E337, ASTM D4414, ASTM D4417, ASTM D4285, ASTM D5161, ASTM D5162, SSPC-AB 1, SSPC-AB 2, SSPC-AB 3, SSPC-SP 10, SSPC-PA 2, SSPC-VIS 1, etc. The criteria for acceptance of wrapping or coatings will be based on the verification of all inspection and hold points established for each phase of the coating work operations. As a minimum, the project specification will require the verification of the following inspection points: pre-surface preparation, surface preparation, compressed air supply, abrasives type and cleanliness, ambient conditions, coating application, wet film thickness, appearance between coats, dry film thickness, holiday (discontinuities in the wrapping or coatings) test, repairs of defects, appearance of finish product and final cure.

Carbon steel buried pipes will be tested prior to backfill in accordance with ASTM standard D5162, "Standard Practice for Discontinuity (Holiday) Testing of Nonconductive Protective Coating on Metallic Substrates" to assure that the wrapped or coated buried pipe is virtually free of discontinuities, bare spots (uncoated), and that exterior surfaces of buried pipes are properly coated.

In-service inspection of buried piping will be performed when exposed during an excavation for any reason.

Part 4

Geomembranes and related waterproofing systems will be installed in conformance with the manufacturers' requirements and quality control/assurance plans which will be prepared for the project. The quality assurance provisions will impose manufacturing, material acceptance, and installation requirements. In particular, full-time, on-site monitoring of geomembrane installation, non-destructive leak testing of field seams, and detailed inspection of the completed installation will be required prior to subsequent backfill placement. Additional details on the installation and inspection procedures are discussed in response to Question 03.08.04-4, Part 1.

When below-grade concrete is excavated for any reason, exposed geomembranes and related waterproofing systems will be inspected.

In addition, the performance of the geomembrane envelopes as barriers to prevent entry of the groundwater will be monitored following construction completion using the monitoring system.

Part 5

Below-grade concrete embedded in non-aggressive fill obtained from off-site and not immersed in aggressive Surficial aquifer groundwater will not experience prolonged exposure to aggressive soil or aggressive groundwater. This concrete will therefore be subjected to examination of the exposed surfaces whenever excavated for any reason, pursuant to SRP 3.8.5. For below-grade concrete foundations immersed in aggressive (low pH) groundwater, a monitoring system will be used to detect any leakage through the geomembrane. Leakage will be removed by portable pumps so as to assure that the concrete does not suffer prolonged exposure to aggressive groundwater.

The in-service inspection program and performance monitoring will be designed and conducted in conformance with the requirements of 10 CFR 50.65, Regulatory Guide 1.160, and Regulatory Guide 1.127.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 144

Question 03.08.04-13

Calvert Cliffs Unit 3 FSAR Table 3E.4-1 presents the governing design load combinations for the UHS Makeup Water Intake Structure and UHS Electrical Building. Provide the following information related to the load definition and load combinations for these site-specific structures:

1. Confirm that all of the load definitions for these site-specific structures are the same as those defined in the US EPR FSAR.
2. Confirm that the methods utilized to determine the individual loads are consistent with the approach used in the US EPR FSAR and provide the magnitude of the live load and snow load for these site-specific structures.
3. Explain why the load combinations in Table 3E.4-1 are considered to bound all of the other load combinations tabulated in the US EPR FSAR.
4. Confirm that for every load combination, where any load reduces the effects of other loads, a load factor of zero is applied/considered for that load.
5. For the stability evaluation load combinations 6 through 8, confirm that the effects due to the buoyancy force based on the maximum groundwater elevation and permanent surcharge loads (of adjacent structure(s)) are also considered.

Response

CCNPP Unit 3 FSAR Table 3E.4-1 on design load combinations has been removed. Structural loads and load combinations for the design of site-specific Seismic Category I structures are specified in revised CCNPP Unit 3 FSAR Sections 3.8.4.3.1 and 3.8.4.3.2, respectively. Additional load combinations for stability evaluation and bearing pressure calculation are specified in revised CCNPP Unit 3 FSAR Section 3.8.5.3.

Part 1

Other than the site-specific loads defined in the revised CCNPP Unit 3 FSAR Section 3.8.4.3.1, namely, probable maximum hurricane (PMH) and the standard plant hurricane (SPH), and the exceptions identified in the response to Question 03.08.04-2 Part 3, it is confirmed that the load definitions for the subject site-specific Seismic Category I structures are the same as those defined in the US EPR FSAR.

Part 2

Other than the site-specific hurricane loads, it is confirmed that the methods utilized to determine the individual loads are consistent with the approach used in the US EPR FSAR. The methodology for calculating SPH and PMH loads is described in the response to Question 03.08.04-2 Part 1.

For the site-specific Seismic Category I structures, the design live load due to rain, snow and ice is based on the normal and extreme winter precipitation events described in FSAR Section 2.3.1.2.2.12. The floor live loads are 100 psf on concrete floors and steel grating floors and platforms per US EPR FSAR Section 3.8.4.3.1.

Part 3

As stated in revised CCNPP Unit 3 FSAR Section 3.8.4.3.2, load combinations for design of site-specific Seismic Category I concrete structures are based on those specified in the U.S. EPR FSAR Section 3.8.4.3.2. In addition, load combinations are also defined for site-specific SPH and PMH loads.

There are four service load combinations presented in the U.S. FSAR Section 3.8.4.3.2 for concrete structures. As stated in the response to Question 03.08.04-5, thermal load effects do not apply. Therefore, the last two service load combinations presented in the U.S. EPR FSAR Section 3.8.4.3.2 for concrete structures are enveloped by the first two service load combinations.

There are four factored load combinations presented in the U.S. EPR FSAR Section 3.8.4.3.2 for concrete structures. The first two load combinations are considered for design of site-specific Seismic Category I concrete structures, with thermal load effects (T_o) neglected. Since there is no postulated high-energy pipe break accident for site-specific Seismic Category I concrete structures, the last two factored load combinations listed in U.S. EPR FSAR Section 3.8.4.3.2 are not applicable.

Stability load combinations are presented in the U.S. EPR FSAR Section 3.8.5.3. These load combinations are used to evaluate the stability of site-specific Seismic Category I structures. In addition, load combinations pertaining to SPH and PMH loads are defined in revised CCNPP Unit 3 FSAR Section 3.8.5.3.

Therefore, the load combinations considered for the site-specific Seismic Category I concrete structures bound the load combinations identified in the U.S. EPR FSAR for other Seismic Category I concrete structures.

Part 4

ACI 349-01, Section 9.2.3 requires that if any load reduces the effects of other loads, the corresponding load factor is taken as 0.9 if the load is always present or occurs simultaneously with the other loads. Otherwise, the factor for that load is taken as zero. This requirement is appropriately addressed for the integrated intake structures.

Part 5

The effects of buoyancy force based on the groundwater elevation of 3 feet and the lateral soil pressure caused by permanent surcharge loads from the adjacent structures or a minimum of 500 psf are considered in the stability evaluation of the site-specific Seismic Category I UHS structures.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 144

Question 03.08.04-14

Calvert Cliffs Unit 3 FSAR Section 3E.4 provides a description of the analysis and design of the UHS Makeup Water Intake Structure and UHS Electrical Building and some limited information about the results in terms of demand member forces for several critical sections (basemats and walls). For the most critical concrete members in the basemat and walls for the UHS Makeup Water Intake Structure and the UHS Electrical Building, provide the resulting member forces (membrane forces, shears, and moments) and comparisons to the section strengths, at least for the most critical governing load combination(s). This information would show the level of margin existing in the design. To facilitate the review, such information is usually presented in tables. Include in these tables the steel areas provided which correspond to the tabulated section strengths.

Response

As stated in updated CCNPP Unit 3 FSAR Sections 3.7.1 and 3.7.2[1], the Ultimate Heat Sink (UHS) Makeup Water Intake Structure is now part of an integrated system of structures which also includes the Forebay and the Circulating Water System Makeup Water Intake Structure, referred to as Common Basemat Intake Structures (CBIS). Dynamic and static analyses of the CBIS and the UHS Electrical Building have been performed and revised CCNPP Unit 3 FSAR Sections 3.8.4, 3.8.5 and 3E.4 revised for the latest analysis and design information.

Member forces for design load combinations for the critical concrete sections of the UHS Makeup Water Intake Structure, Forebay and UHS Electrical Building are provided in revised CCNPP Unit 3 FSAR Tables 3E.4-1 through 3E.4-4. Section strength, demand/capacity ratios, and corresponding steel reinforcement are also presented in these tables.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

Enclosure 2

Response to NRC Request for Additional Information

RAI 145, Foundations,
Questions 03.08.05-01, 03, 04, 05 and 06,

Calvert Cliffs Nuclear Power Plant, Unit 3

RAI 145

Question 03.08.05-1

The U.S. EPR FSAR states that "A COL applicant that references the U.S. EPR design certification will evaluate site-specific methods for shear transfer between the foundation basemats and underlying soil for soil parameters that are not within the envelope specified in Section 2.5.4.2." Calvert Cliffs Unit 3 FSAR Section 3.8.5.5 – Structural Acceptance Criteria, provides an explanation of how this COL Item is addressed. Provide the requested information to address the following items:

1. Section 3.8.5.5 lists three bulleted items that participate in resisting sliding of the Emergency Power Generating Buildings (EPGBs) and the Essential Service Water Buildings (ESWBs). Explain why these items were listed only for the EPGBs and the ESWBs and not for the Nuclear Island (NI), or for Seismic Category II and II-SSE structures. Is the methodology used for the EPGBs and ESWBs different than that used for the NI? Explain why the list of three bulleted items does not include: (1) the resistance to sliding between the mud mat and waterproofing membrane, and (2) shear resistance within the soil.

2. In order to achieve a coefficient of friction between the basemat and the mud mat of 0.7 will the concrete surface of the mud mat be required to be intentionally roughened in accordance with ACI 349-97 Section 11.7? If not, then demonstrate that the coefficient of friction between the basemat and the mud mat is equal to at least 0.7.

3. Section 3.8.5.6.1 of the EPR FSAR indicates that the textured waterproofing membrane in the mud mat beneath the basemat will have a coefficient of friction of at least 0.7 and that this will be demonstrated by vendor testing. Where is this requirement stated in the Calvert Cliffs Unit 3 FSAR, and where are the vendor test data results presented?

4. Section 3.8.5.5 of the Calvert Cliffs FSAR indicates that a coefficient of friction of 0.70 at the soil-soil interface beneath the EPGB and ESWB basemats cannot be achieved for the existing underlying soils. Therefore, during excavation of the soil at the site, additional soil material will be removed below the structures and structural backfill material will be placed. Section 3.8.5.5 further states that the coefficient of friction for the actual structural backfill material will be confirmed to meet the EPR FSAR requirement of 0.70 prior to placement of the structural backfill. Based on the information in Sections 3.8.5.5, 2.5.4.5.2 and Figures 2.5-130 through 2.5-134, there appears to be only 4 feet of structural backfill material that will be used under several of the structures (e.g., Reactor Containment Building, Safeguards Building, and Fuel Building). Explain how this depth is determined and why is this considered to be sufficient to preclude sliding/soil failure beneath the 4 foot structural backfill layer. Explain how it will be determined that the required coefficient of friction is met for the critical soil layer prior to placement of the structural backfill. What type of testing will be performed to determine the coefficient of friction at both the soil-soil and soil-concrete interfaces? When will this be performed? Also, the ITAAC in Application Part 10, Table 2.4-1, related to demonstrating the coefficient of friction for the various structures do not clearly state that the coefficient of friction of 0.70 will be demonstrated for the soil-soil and soil-concrete interfaces.

5. The last paragraph of Section 3.8.5.5 states "Coefficients of friction at the soil-soil and soil-concrete interfaces are consistent with the values in Section 2.5.4.10.2, including Table 2.5-

36." Explain the meaning of this sentence. Are there test data at this time which demonstrate that the coefficients of friction at the soil-soil and soil-concrete interfaces will be at least 0.70 as required? Why is Section 2.5.4.10.2 referenced since it addresses settlement with no discussion about coefficients of friction? Although Table 2.5-36 provides coefficients of sliding, the coefficients correspond to values far below the requirement of 0.70.

Response

Part 1

Site-specific stability evaluations have been performed for the Nuclear Island (NI) common basemat structures, the Emergency Power Generating Buildings (EPGBs), and the Essential Service Water Buildings (ESWBs) using the methodology described in updated CCNPP Unit 3 FSAR Section 3.7.2.14[1]. The results are discussed in revised CCNPP Unit 3 FSAR Section 3.8.5.5. The list of the shear transfer mechanism has been updated. The results conclude that shear transfer from the superstructure to the foundation media across various interfaces can be achieved without utilizing passive soil pressures.

Revised CCNPP Unit 3 FSAR Section 3.8.5.5 includes discussion on shear transfer at the waterproofing membrane interfaces. See the response to Part 3 of this question below for the frictional characteristics of waterproofing membrane. Shear resistance within the soil layers below the foundation basemat including the structural fill is addressed in updated CCNPP Unit 3 FSAR Section 2.5.4[6] in terms of internal friction angle and friction coefficient.

Discussion on Seismic Category II and II-SSE structures are not presented in the CCNPP Unit 3 FSAR since the FSAR is prepared according to the standard format and content presented in RG 1.206, which has no requirement for such information for non-safety related structures.

Part 2

The surface of the mud mat will not be intentionally roughened. Instead, the basemat will be cast directly on the hardened mud mat. According to ACI 349-01 Section 11.7, this provides a coefficient of friction of 0.6. Site specific seismic stability evaluation shows that the required minimum coefficients of friction under safe shutdown earthquake loading (SSE) loading to achieve factor of safety (FOS) = 1.1 are as follows:

- NI Common Basemat Structures: 0.28
- EPGBs: 0.32
- ESWBs: 0.18

Therefore, a coefficient of friction of 0.6 between the basemat and the mud mat is adequate for the NI common basemat structures, the EPGBs, and the ESWBs under the CCNPP Unit 3 site-specific loading conditions. A departure from the U.S. EPR FSAR is required since the coefficient of friction is less than 0.70.

Part 3

As discussed in the response to RAI 144 Question 03.08.04-4, waterproofing membrane is utilized to protect foundation basemats subject to low-pH groundwater. The waterproofing membrane is envisioned to be placed between two layers of compacted sand (complying with ASTM C33), as illustrated in revised CCNPP Unit 3 FSAR Figure 3.8-6.

The design coefficient of friction for the interface of the waterproofing geomembrane with sand has been established based on available data from GRI Report #30 by Koerner and Narejo (2005) [18]. This report shows that the peak and residual coefficients of friction on the interface between textured HDPE geomembrane and sand, based on the aggregated results of more than 250 individual tests, are 0.67 (tan 34°) and 0.60 (tan 31°), respectively. Stability analyses have therefore been performed using a more conservative interface friction coefficient of 0.52 (tan 27.5°). Revised CCNPP Unit 3 FSAR Section 3.8.5.5 includes this information. Vendor testing will be performed prior to construction to confirm the coefficient of friction used during analysis and design. Vendor testing of the geomembrane is added as a new table, Table 2.4-37, in COLA Part 10 – ITAAC.

Part 4

Revised CCNPP Unit 3 FSAR Figure 3.8-6 shows the conceptual configuration of waterproofing membrane along with various interface layers beneath the basemat of the Nuclear Island Common Basemat Structures. Potential sliding interfaces below the NI structure foundation basemat and the associated friction coefficients are as follows:

- Basemat/mud mat: 0.60
- Mud mat/sand: 0.58
- Sand/waterproofing membrane: 0.52
- Sand/structural fill: 0.58
- Structural fill/Soil stratum IIb: 0.47 (plus 1 ksf adhesion)

Discussions on the friction coefficients between basemat and mud mat and between sand and waterproofing membrane are provided in the responses to Parts 2 and 3 of this question. The coefficient of friction between mud mat and sand is based on the internal friction angle of sand established by a range of direct shear tests[19]. Coefficients of friction at the Sand/Backfill and Backfill/Soil stratum IIb interfaces are determined based on the smaller internal friction angle of the two interfacing soil layers. Internal friction angles of various in-situ soil layers and the structural fill are based on the laboratory tests described in updated CCNPP Unit 3 FSAR Section 2.5.4[6].

The sliding demand of the NI common basemat structures, in terms of minimum required friction

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18 Koerner, G.R., and D. Narejo (2005). "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces," GRI Report #30, Geosynthetic Research Institute, Folson, PA (p. 53).

19 Berney, E.S. IV, and Smith, D.M., Mechanical and Physical Properties of ASTM C33 Sand, U.S. Army Corps of Engineers, Engineering Research and Development Center, Vicksburg, MS, ERDC/GSL TR-08-2, February 2008

coefficient, is 0.28 under SSE seismic load, which is well below the available frictional resistance. Based on this demand, the minimum factor of safety against sliding is 1.88.

Table 2.4-1 in COLA Part 10 – ITAAC was revised in January 2010[20]. The revision removed confirmation of soil parameters. A new ITAAC to confirm the coefficient of friction of the membrane has been created as discussed in Part 3.

Part 5

The referenced text "*Coefficients of friction at the soil-soil and soil-concrete interfaces are consistent with the values in Section 2.5.4.10.2, including Table 2.5-36.*" has been removed from CCNPP Unit 3 FSAR Section 3.8.5.5. Response to this question is based on the updated revision of the FSAR.

As discussed in the response to Part 4 of this question, internal friction angles of in-situ soil and backfill material (including sand) are determined based on laboratory tests. Coefficients of friction at soil-soil interfaces are determined based on the smaller internal friction angle of the two interfacing soil layers. The coefficients of friction at the concrete-soil/soil-soil interfaces are summarized in revised CCNPP Unit 3 FSAR Table 3.8-1.

Though the coefficients of friction available for sliding resistance shown in revised CCNPP Unit 3 FSAR Table 3.8-1 are smaller than 0.70, site-specific sliding stability evaluations for the NI common basemat structures, the EPGBs, and the ESWBs have been performed to demonstrate that the governing factors of safety, reported in FSAR Section 3.8.5.5, exceed the minimum allowable values presented in SRP 3.8.5 Acceptance Criteria II.5.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

Enclosure 4 provides a departure for the coefficient of friction that is to be added to COLA Part 7.

Enclosure 5 provides a new ITAAC Table 2.4-37 for COLA Part 10.

Enclosure 6 provides conforming changes to CCNPP Unit 3 FSAR Section 1.8.2 to reflect the new departure.

[20] UN#10-027, UniStar Nuclear Energy Letter from Greg Gibson to Document Control Desk, U.S. NRC, Shear Wave Velocity Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) Update and Departure, dated January 29, 2010.

RAI 145

Question 03.08.05-3

Calvert Cliffs Unit 3 FSAR Table 3.8-1 provides a summary table for evaluation of the UHS Makeup Water Intake Structure basemat for soil bearing pressure and stability evaluation (sliding and overturning). Provide the information requested below related to this table.

1. Define the various load combinations applicable to all of the entries in FSAR Table 3.8-1. How do these load combinations compare with those in NRC SRP 3.8.5?

2. For the site-specific UHS Makeup Water Intake Structure, provide a description and the results of the evaluation performed to demonstrate that the sliding, overturning, and flotation load combinations meet the acceptance criteria presented in NRC SRP 3.8.5. Include an explanation of how the demand (applied) SSE loading was developed for horizontal shear force and overturning moment, how the resisting forces for shear and overturning were determined, whether two sets of two-dimensional (2-D) calculations were performed (i.e., evaluations performed for NS and vertical, and then EW and vertical), whether upward vertical SSE force was assumed to reduce dead weight, and whether buoyancy was considered.

3. What was the governing coefficient of friction that was used in these stability evaluations (basemat to mud mat, mud mat to waterproofing membrane, mud mat to soil, or soil to soil (which could vary from 0.35 to 0.7 depending on whether the soil is existing soil from the site or structural backfill)).

4. Section 3.8.5.5 indicates that passive earth pressure and shear keys are utilized to transfer shear into the soil. To develop the passive earth pressure of the soil, the foundation would need to displace sufficiently to mobilize the soil passive resistance. Thus, the dynamic coefficient of friction would be more appropriate than the static coefficient of friction (which would have a smaller value). Explain whether a dynamic coefficient of friction is utilized and the magnitude of the governing dynamic coefficient of friction, or provide the technical basis for using the static coefficient of friction.

5. Provide a complete description and results of the stability evaluation for the UHS Electrical Building. Also, provide the soil bearing, settlement, and stability evaluations for the Seismic Category II and II-SSE site-specific structures.

Response

The Ultimate Heat Sink (UHS) Makeup Water Intake Structure, Forebay, and CWS Makeup Water Intake Structure are now integrated and known as the Common Basemat Intake Structures (or CBIS) since they share a common basemat. Stability evaluation of the structures has been performed for the Common Basemat Intake Structures and UHS Electrical Building. The results are provided in revised CCNPP Unit 3 FSAR Table 3.8-2. FSAR Table 3.8-1 now presents information on friction parameters.

Part 1

Revised CCNPP Unit 3 FSAR Table 3.8-2 shows the results of the stability evaluation for various load combinations. The load combinations for stability evaluation are defined in the U.S. EPR FSAR Section 3.8.5.3. These load combinations are the same as those in SRP 3.8.5 Acceptance Criteria II.3, except that the operational basis earthquake (OBE) load combination is not explicitly considered based on the discussion in revised CCNPP Unit 3 FSAR Section 3.8.4.3. Additional load combinations are defined in revised CCNPP Unit 3 FSAR Section 3.8.5.3 for site-specific hurricane events. Note that flotation load case is evaluated for buoyancy under normal operation and hurricane conditions.

Part 2

Procedures to evaluate sliding, overturning, and flotation of the CBIS are presented in revised CCNPP Unit 3 FSAR Section 3.8.5.4.6. The summary of stability evaluation is provided in revised CCNPP Unit 3 FSAR Table 3.8-2.

Methods to evaluate the safe shutdown earthquake (SSE) seismic sliding and overturning demands, including effects of earthquake motions in three orthogonal directions, are discussed in updated CCNPP Unit 3 FSAR Section 3.7.2.14.2[1].

The resistance to sliding is provided by the friction between the basemat and the mud mat, friction and adhesion between the mud mat and Soil Layer IIC (Chesapeake clay/silt), and friction (traction) between the subsurface side walls and adjacent backfill. Normal force from loads contributing to the structural mass used in the SASSI 2000 analysis (including 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 reduced by the upward seismic forces and buoyancy in the calculation of frictional resistance. Sliding is checked at each time step for each soil profile and the governing factor of safety is determined. The governing factor of safety against sliding under SSE loading is 1.18 at the interface of basemat and mud mat.

Resistance to overturning is provided by the loads contributing to the structural mass used in the SASSI 2000 analysis (including 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), reduced by buoyancy. Overturning is checked independently for each of the four basemat edges for each time step and the governing factors of safety are determined. The governing factor of safety against overturning under SSE loading is 2.04.

Factor of Safety for sliding, overturning and flotation for various load combinations are reported in revised CCNPP Unit 3 FSAR Table 3.8-2, which shows that the required minimum factors of safety presented in SRP 3.8.5 Acceptance Criteria II.5 are met.

Part 3

The frictional parameters for various sliding interfaces are given in revised CCNPP Unit 3 FSAR Table 3.8-1 and summarized below:

- Between basemat and mud mat: coefficient of friction = 0.6
- Between mud mat and in-situ Type IIC soil: coefficient of friction = 0.21; adhesion = 1.2 ksf
- Between sidewalls and structural fill: coefficient of friction (traction) = 0.58

The coefficient of friction of 0.70 proposed by the U.S. EPR FSAR cannot be achieved across the various interfaces. Therefore, a departure is required as described in the response to Question 03.08.05-1. A waterproofing membrane is not used for CBIS.

Part 4

Shear keys are not used in the current configuration of the CBIS. Sufficient resistance can be developed at the various interfaces to achieve the minimum factor of safety against sliding as shown in revised CCNPP Unit 3 FSAR Table 3.8-2. Passive earth pressure is not relied upon to maintain the stability of the CBIS. Subsequently, since no sliding occurs, static friction coefficients are used. Revised CCNPP Unit 3 FSAR Section 3.8.5.5 reflects these conclusions.

Part 5

The same procedures as described in the response to Part 2 above are used to perform the stability evaluations of the UHS Electrical Building except that passive earth pressure is utilized to provide sliding resistance during a SSE or PMH event.

The UHS Electrical Building utilizes a waterproofing membrane to minimize water intrusion into the below grade compartment. The resistance to sliding during a SSE or PMH event is provided by friction across various interfaces and a portion of the available passive resistance. When the passive resistance is utilized, all the static friction coefficients are reduced by 25% to approximate the dynamic coefficients of friction. Stability evaluation indicates that a maximum of approximately 30% of full passive resistance is needed for the most critical loading condition (SSE) to achieve a minimum factor of safety (FOS) = 1.1 resulting in an estimated movement of approximately ¼ inch. The utility connections to the Electrical Building will be designed to accommodate this movement. The results of stability evaluation are presented in revised CCNPP Unit 3 FSAR Table 3.8-2. Note that passive resistance is used only for load combinations (3) and (6) in Table 3.8-2 for the UHS Electrical Building.

Discussions on Seismic Category II and II-SSE structures are not presented in the CCNPP Unit 3 FSAR since the FSAR is prepared according to the standard format and content presented in RG 1.206, which has no requirement for such information for non-safety related structures.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

Enclosure 6 contains a minor markup to the updated CCNPP Unit 3 FSAR Section 3.7.2.14.

RAI 145

Question 03.08.05-4

Calvert Cliffs Unit 3 FSAR Section 3.8.5.5.1 for the NI, 3.8.5.5.2 for the EPGBs, and Section 3.8.5.5.3 for the ESWBs acknowledge that there are some differences from the U.S. EPR standard plant in the soil bearing pressures, stresses in the base mat, and stability evaluations due to site-specific settlements and groundwater conditions. The extent of these differences is sometimes identified as negligible, within allowable values, or less than the corresponding section capacity. For each of these structures, quantify the specific differences from the U.S. EPR standard plant discussed in FSAR Sections 3.8.5.5.1, 3.8.5.5.2 and 3.8.5.5.3 rather than using qualitative terms.

FSAR Section 3.8.5.5.2 for the EPGBs includes a statement that the "Factors of safety against sliding and overturning remain within allowable values" and Section 3.8.5.5.3 for the ESWBs has a similar statement which indicates that the effects are "negligible." No such discussion is given for the NI. Due to the increased site-specific settlements and higher groundwater elevations, and changes in soil bearing pressures, coefficient of frictions, and soil properties from the values specified in the EPR FSAR, provide a description and the results of the stability evaluations for the NI, EPGBs, and ESWBs. If the differences in the responses of the structures from the U.S. EPR standard plant are truly negligible eliminating the need for any of the specific stability evaluations, provide the technical justification including the quantitative data to support the conclusion.

How has the potential effect of saturated soils from groundwater been considered in (1) the calculation of the subgrade modulus/soil spring stiffness used in the various analyses, (2) all seismic soil structure interaction (SSI) analyses for development of building loads and displacements, (3) calculations for soil bearing pressure demand, (4) stability evaluations (including coefficient of friction and passive pressure), and (5) design of the basemat foundation and walls?

Response

Part 1

Site-Specific differences from U.S. EPR FSAR standard plant for the EPGBs and ESWBs are described below:

a. Groundwater Elevation

The highest post-construction groundwater level in the powerblock area is about 30 ft below the finished grade. This water table is well below the U.S. EPR FSAR design groundwater level of 3.3 ft below the finished grade. Therefore, the two instances – under Essential Service Water Building (ESWB) 1 and under the Emergency Power Generating Building (EPGB) 1/2 – where the predicted groundwater table exceeded the U.S. EPR FSAR certified design groundwater level do not exist in the post-construction environment. In fact, the groundwater level will be below the foundation basemats of both the EPGBs and ESWBs. Therefore, the discussion on the effects of higher groundwater elevation is not applicable and has been removed from revised CCNPP Unit 3 FSAR Sections 3.8.5.5.2 and 3.8.5.5.3.

b. Differential Settlement

The results of the updated settlement analysis are described in FSAR Section 2.5.4.10.2[6].

For the EPGBs, the conventional geotechnical analysis considering the foundation as a flexible plate estimates a maximum differential settlement of 1/1166, which is about 3% higher than the allowable value of 1/1200 described in the U.S. EPR FSAR. Therefore, an evaluation was conducted to determine the potential effect of increased differential settlement on the foundation mat. Considering a conservative differential settlement value of 1/550 (based on overall building tilt), the analysis concluded that increase in the basemat flexural response is less than 3%. Therefore, EPGB basemat is structurally adequate to resist the increase in the flexural responses that would result from the estimated differential settlement. Additionally, the effect of the stiffness of a 6 ft thick reinforced concrete basemat on differential settlement was evaluated using a finite element analysis based on the CCNPP Unit 3 site-specific soil springs. This analysis resulted in an estimated maximum differential settlement of 1/2714. The differential settlements derived from the geotechnical settlement analysis based on a flexible foundation are much greater (more conservative) than those that are anticipated for a stiffened basemat. Discussion on the effects of increased differential settlement on EPGB design has been provided in revised CCNPP Unit 3 FSAR Section 3.8.5.5.2.

For the ESWBs, the conventional geotechnical analysis considering the foundation as a flexible plate estimates a maximum differential settlement of 1/845, which exceeds the allowable value of 1/1200 described in the U.S. EPR FSAR. Therefore, an evaluation was conducted to determine the potential effect of increased differential settlement on the foundation mat. Considering a conservative differential settlement value of 1/600 (based on overall building tilt), the analysis concluded that increase in the basemat flexural response is less than 5%. Therefore, ESWB basemat is structurally adequate to resist the increase in the flexural responses that would result from the estimated differential settlement. Additionally, the effect of the stiffness of a 6 ft thick reinforced concrete basemat on differential settlement was evaluated using a finite element analysis based on the CCNPP Unit 3 site-specific soil springs. This analysis resulted in an estimated maximum differential settlement of 1/1417. The differential settlements derived from the geotechnical settlement analysis based on a flexible foundation are much greater (more conservative) than those that are anticipated for a stiffened basemat. Discussion on the effects of increased differential settlement on ESWB design has been provided in revised CCNPP Unit 3 FSAR Section 3.8.5.5.3.

c. Coefficient of Friction

U.S. EPR FSAR Table 2.1-1 identifies a value of 0.70 as coefficient of static friction at the soil basemat interface. A departure from the U.S. EPR FSAR is required since the coefficient of friction is less than 0.70. However, the results of the site-specific sliding stability evaluation (reported in Table 1 below) indicate that the available factors of safety against sliding exceed the required minimum value of 1.1, as specified by NUREG 0800, Standard Review Plan 3.8.5, Structural Acceptance Criteria II.5. Additional discussion on the coefficients of friction is provided in the response to Question 03.08.05-1.

Part 2:

Site-specific stability evaluation for SSE loading based on the results of site-specific soil-structure Interaction (SSI) analysis is performed using the methodologies described in updated CCNPP Unit 3 FSAR Section 3.7.2.14[1]. Table 1 provides the factors of safety (FOS) against sliding and overturning. Passive soil pressure is not utilized in the evaluations. Note that the calculated factors of safety satisfy the requirement stipulated in SRP 3.8.5 Acceptance Criteria II.5.

Table 1 FOS against Sliding and Overturning

Load Case: D+H+E' (based on SRP 3.8.5 Acceptance Criteria II.3)		
Building	Sliding	Overturning
EPGB	1.77	3.17
ESWB	3.19	4.0

Site-specific wind, tornado, and flood loads are enveloped by the corresponding U.S. EPR FSAR design values, as shown in CCNPP Unit 3 FSAR Table 2.0-1. The site-specific wind and tornado loads are also enveloped by the site-specific SSE loading. Therefore, explicit stability evaluation of the EPGBs and ESWBs due to site-specific wind, tornado, and flood loading is not performed.

Part 3:

Since the post-construction groundwater table is below the EPGB and ESWB foundation basemat, potential effects of saturated soils are not applicable to these structures. The moist soil density is used for site-specific seismic reconciliation and stability evaluations of the EPGBs and ESWBs.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

Enclosure 4 contains an update to the departure on ESWB and EDGB differential settlement.

RAI 145

Question 03.08.05-5

For the UHS Makeup Water Intake Structure finite element model, Calvert Cliffs Unit 3 FSAR Section 3E.4.1 states that "Pinned supports are placed at all nodes of the base mat. During detailed engineering, and upon completion of the Final Geotechnical Site Investigation, it will be confirmed that the use of soil springs (in lieu of pinned supports) does not adversely affect the design results." This type of statement, which relies on future geotechnical site investigations, appears in several other locations in Section 3.8 of the FSAR (e.g., Sections 3.8.4.3, 3.8.4.4.5, 3.8.4.5, 3.8.5.5.4). Explain why such assumptions are necessary rather than utilizing bounding/conservative assumptions. When would these future geotechnical site investigations be performed?

Response

Final geotechnical site investigations have been performed and the results are provided in updated CCNPP Unit 3 FSAR Section 2.5.4[6]. As stated in updated CCNPP Unit 3 FSAR Sections 3.7.1 and 3.7.2[1], the Ultimate heat Sink (UHS) Makeup Water Intake Structure is now a part of an integrated system of structures which also includes the Forebay and the Circulating Water System Makeup Water Intake Structure, referred to as Common Basemat Intake Structures (CBIS). Static analysis of the CBIS has been performed using finite element model that incorporates springs at the basemat nodes (in lieu of the pinned supports) based on the site-specific soil parameters. Revised CCNPP Unit 3 FSAR Sections 3.8.4, 3.8.5 and Appendix 3E include discussion of the model and other aspects of analysis and design of the CBIS.

Confirmatory statements such as "*Pinned supports are placed at all nodes of the base mat. During detailed engineering, and upon completion of the Final Geotechnical Site Investigation, it will be confirmed that the use of soil springs (in lieu of pinned supports) does not adversely affect the design results*", which relied on future geotechnical site investigations, have been deleted from revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E. Therefore these assumptions are no longer necessary.

COLA Impact

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

RAI 145

Question 03.08.05-6

Calvert Cliffs Unit 3 FSAR Section 3E.4.1 – Base Mat of the UHS Makeup Water Intake Structure, under the heading Results of Critical Section Design, states that:

> "The mat dimensions used in the seismic analysis are based on the building periphery and not the extended base mat. Thus, the maximum difference between the base mat dimension in soil contact and the corresponding mat dimension used in the dynamic analysis is 8 ft (2.4 m), or approximately 15 percent of the overall mat dimension. During detailed engineering, it will be confirmed that the mat extensions do not adversely impact the accelerations and in-structure response spectra generated via the seismic analysis."

This statement indicates that the analysis and design of the site-specific structures presented in the FSAR do not correspond to the actual configuration that will be constructed. Therefore, include in the FSAR the description and results of the analysis and design of the site-specific structures that match the actual configurations that will be constructed. If this is not done, then provide a sufficient technical basis supported by quantitative data to demonstrate the adequacy of the existing analysis and design.

Response

The above statement in the CCNPP Unit 3 FSAR was based on the preliminary stability calculations using the original configuration and layout of the Ultimate heat Sink (UHS) Makeup Water Intake Structure. This structure is now a part of a larger integrated system of structures that includes Forebay and Circulating Water System Makeup Water Intake Structure. The new configuration of the structure does not include the basemat extensions. Seismic analysis of the new configuration is presented in updated CCNPP Unit 3 FSAR Section 3.7[1].

COLA Impact:

The information presented in this response is included in the revised CCNPP Unit 3 FSAR Section 3.8 and Appendix 3E provided in Enclosure 3.

Enclosure 3

Markup of FSAR Section 3.8,
Design of Category I Structures, Calvert Cliffs Nuclear Power Plant, Unit 3

Calvert Cliffs Nuclear Power Plant, Unit 3

3.8 DESIGN OF CATEGORY I STRUCTURES

This section of the U.S. EPR FSAR is incorporated by reference with the departures and supplements as described in the following sections.

3.8.1 CONCRETE CONTAINMENT

No departures or supplements.

3.8.1.1 Description of the Containment

No departures or supplements.

3.8.1.2 Applicable Codes, Standards, and Specifications

No departures or supplements.

3.8.1.3 Loads and Load Combinations

The U.S. EPR FSAR includes the following COL Item in Section 3.8.1.3:

> A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard plant design envelope for the RCB, or perform additional analyses to verify structural adequacy.

This COL Item is addressed as follows:

{The RCB design for CCNPP Unit 3 is the standard RCB design as described in the U.S. EPR FSAR without departures. Site-specific loads are confirmed to lie within the standard U.S. EPR design certification envelope. Site-specific seismic, RSB, and buoyancy conditions are addressed in Sections 3.7.2, 3.8.4, and 3.8.5, respectively.}

3.8.1.4 Design and Analysis Procedures

No departures or supplements.

3.8.1.5 Structural Acceptance Criteria

No departures or supplements.

3.8.1.6 Materials, Quality Control, and Special Construction Techniques

No departures or supplements.

3.8.1.6.1 Concrete Materials

No departures or supplements.

3.8.1.6.2 Reinforcing Steel and Splice Materials

No departures or supplements.

3.8.1.6.3 Tendon System Materials

No departures or supplements.

3.8.1.6.4 Liner Plate System and Penetration Sleeve Materials

No departures or supplements.

3.8.1.6.5 Steel Embedments

No departures or supplements.

3.8.1.6.6 Corrosion Retarding Compounds

No departures or supplements.

3.8.1.6.7 Quality Control

The QA program for this section is discussed in Section 3.1.1.1.1.

3.8.1.6.8 Special Construction Techniques

No departures or supplements.

3.8.1.7 Testing and Inservice Inspection Requirements

No departures or supplements.

3.8.2 STEEL CONTAINMENT

No departures or supplements.

3.8.3 CONCRETE AND STEEL INTERNAL STRUCTURES OF CONCRETE CONTAINMENT

3.8.3.1 Description of the Internal Structures

No departures or supplements.

3.8.3.2 Applicable Codes, Standards, and Specifications

No departures or supplements.

3.8.3.3 Loads and Load Combinations

The U.S. EPR FSAR includes the following COL Item in Section 3.8.3.3:

> A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard design envelope for RB internal structures, or perform additional analyses to verify structural adequacy.

This COL Item is addressed as follows:

{The Reactor Building (RB) (i.e., the Reactor Containment Building (RCB)) internal structural design is the standard design as described in the U.S. EPR FSAR without departures.

Site-specific loads are confirmed to lie within the standard U.S. EPR design certification envelope. Relative site-specific conditions are addressed in Section 3.7.2.}

3.8.3.4 Design and Analysis Procedures

No departures or supplements.

3.8.3.5 Structural Acceptance Criteria

No departures or supplements.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

No departures or supplements.

3.8.3.7 Testing and Inservice Inspection Requirements

No departures or supplements.

3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES

3.8.4.1 Description of the Structures

The U.S. EPR FSAR includes the following COL Items in Section 3.8.4:

> A COL applicant that references the U.S. EPR design certification will describe any differences between the standard plant layout and design of Seismic Category I structures required for site-specific conditions.
>
> A COL applicant that references the U.S. EPR design certification will address site-specific Seismic Category I structures that are not described in this section.

The COL Items are addressed as follows:

{The standard plant layout and design of other Seismic Category I Structures is as described in the U.S. EPR FSAR without departures.

The site-specific Seismic Category I structures at CCNPP Unit 3 are:

- Buried Conduit and Duct banks (Section 3.8.4.1.8).
- Buried Pipe and Pipe Ducts (Section 3.8.4.1.9).
- Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building (Section 3.8.4.1.11).}

3.8.4.1.1 Reactor Shield Building and Annulus

No departures or supplements.

3.8.4.1.2 Fuel Building

No departures or supplements.

3.8.4.1.3 Safeguard Buildings

No departures or supplements.

3.8.4.1.4 Emergency Power Generating Buildings

No departures or supplements.

3.8.4.1.5 Essential Service Water Buildings

No departures or supplements.

3.8.4.1.6 Distribution System Supports

No departures or supplements.

3.8.4.1.7 Platforms and Miscellaneous Structures

No departures or supplements.

3.8.4.1.8 Buried Conduit and Duct Banks

The U.S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.8.4.1.8:

> A COL applicant that references the U.S. EPR design certification will provide a description of Seismic Category I buried conduit and duct banks.
>
> [[Buried conduits are steel while conduits in encased duct banks may be poly-vinyl-chloride (PVC) or steel. Duct banks may be directly buried in the soil; encased in lean concrete, concrete, or reinforced concrete. Concrete or reinforced concrete encased duct banks will be used in heavy haul zones, under roadway crossings, or where seismic effects dictate the requirement. Encasement in lean concrete may be used in areas not subject to trenching or passage of heavy haul equipment, or where seismic effects on the conduit are not significant.]]

{This COL Item is addressed as follows, and the conceptual design information is replaced with site-specific information for CCNPP Unit 3:

Figure 3.8-1 provides an overall site plan of Seismic Category I buried duct banks. The buried duct banks run between the Nuclear Island (NI) and the Intake Structures along the utility corridor. Figure 3.8-2 provides a detail plan of Seismic Category I buried duct banks in the vicinity of the NI. No Seismic Category I buried conduits exist for CCNPP Unit 3.

Seismic Category I buried electrical duct banks traverse from:

- The UHS Makeup Water Intake Structure to the UHS Electrical Building.

- Each Essential Service Water Building to the UHS Electrical Building, including underneath the main heavy haul road.

- The Safeguards Buildings to the four Essential Service Water Buildings and both Emergency Power Generating Buildings.

For the first item, the UHS Makeup Water Intake Structure and UHS Electrical Building are discrete structures housing mechanical and electrical equipment, respectively. Buried electrical duct banks traverse the two structures to provide power to the equipment, including the UHS Makeup Water pumps.

Buried electrical duct banks consist of polyvinyl chloride (PVC) conduit encased in reinforced concrete. In addition to its structural function, the reinforced concrete facilitates maintenance of conduit spacing / separation requirements and protects the conduit.

Where buried safety-related electrical duct banks and the UHS makeup water pipes traversing between the UHS Makeup Water Intake Structure and the four ESWBs need to cross each other, the buried electrical duct banks are located below the pipes to facilitate future pipe maintenance. To facilitate cable pulling and routing, manholes are provided at strategic locations.

In areas where the buried electrical duct banks are below the groundwater table, the joints between buried duct banks and manholes have PVC water stops to prevent water intrusion.

Buried electrical duct banks have drain pipes at the bottom, and are constructed such that they slope from manhole to manhole. The low point manholes have a sump with a pump for collecting and disposing water.

Waterproofing membrane, as described in Section 3.8.4.6.1, is used, as necessary, to protect buried electrical duct banks from the corrosive effects of low-pH groundwater from the Surficial aquifer in the powerblock area.}

3.8.4.1.9 Buried Pipe and Pipe Ducts

The U.S. EPR FSAR includes the following COL Item in Section 3.8.4.1.9:

> A COL applicant that references the U.S. EPR design certification will provide a description of Seismic Category I buried pipe and pipe ducts.

This COL Item is addressed as follows:

{Figure 3.8-3 provides an overall site plan of Seismic Category I buried pipe. Pipes run beneath the final site grade. Buried pipe ducts are not used for CCNPP Unit 3. Two buried CCNPP Unit 3 Intake Pipes run from the CCNPP Unit 3 Inlet Area to the Unit 3 Forebay (See Figure 2.4-51). Four UHS Makeup Water pipes emanate from the UHS Makeup Water Intake Structure and terminate at the ESWBs. These pipes run within the utility corridor, shown in Figure 3.8.3, and pass under the main Haul Road which runs in the East-West direction adjacent to the North side of the CCNPP Unit 3 powerblock.

Figure 3.8-4 provides a detail plan of Seismic Category I buried ESW pipe in the vicinity of the NI. As illustrated in the figure, the Seismic Category I buried ESW piping consists of:

- Large diameter supply and return pipes between the Safeguards Buildings and the ESWBs.
- Large diameter supply and return pipes from the EPGBs which tie in directly to the aforementioned pipes.

Fire Protection pipe traverses from the UHS Makeup Water Intake Structure and UHS Electrical Building to the vicinity of the NI, where a loop is provided to all buildings. In accordance with Section 3.2.1, Fire Protection piping to Seismic Category I structures that is classified as: 1) Seismic Category II is designed to maintain its pressure boundary after an SSE event; and 2) Seismic Category II-SSE is designed to remain functional during and following an SSE event.

The buried piping is directly buried in the soil (i.e., without concrete encasement) unless detailed analysis indicates that additional protection is required. The depth of the soil cover is generally sufficient to provide protection against frost (top surface of the pipe is below the site-specific frost depth), surcharge effects, and tornado missiles. Structural fill is used as bedding material underneath the pipe. As an alternate, lean concrete may be used. Additionally, soil surrounding the pipe is compacted structural fill.}

3.8.4.1.10 Masonry Walls

{No departures or supplements.}

3.8.4.1.11 {Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building}

{This section is added as a supplement to U.S. EPR FSAR Section 3.8.4.1.

The Seismic Category I Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are reinforced concrete structures situated along the western shoreline of the Chesapeake Bay. As illustrated in Figure 9.2-4, the Forebay is connected to the CWS Makeup Water Intake Structure (Seismic Category II) and the Intake Pipes (Seismic Category I) from the north (plant reference) and the UHS Makeup Water Intake Structure from the south. The two intake pipes transport water (under gravitational head) from the Chesapeake Bay to the Forebay, which supplies water to both the CWS Makeup Water Intake Structure and the UHS Makeup Water Intake Structure. The UHS Makeup Water Intake Structure and UHS Electrical Building house components associated with the UHS Makeup Water System, which provides makeup water to the Essential Service Water Cooling Tower basins for extended cooling that starts 72 hours after a design basis accident. Figure 3.8-1 shows the position of the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building relative to the NI.

As illustrated in Figures 9.2-4, 9.2-5, and 9.2-6, the Forebay is a below-grade open top reinforced concrete water basin, with overall dimensions of 109 ft (33.2 m) long by 89 ft (27.1 m) wide by 39 ft (11.9 m) deep, including a 5 ft (1.5 m) thick basemat. Inside dimensions of the Forebay are 100 ft (30.5 m) long by 80 ft (24.4 m) wide, with 4.5 ft (1.4 m) thick walls.

The Forebay is embedded approximately 37.5 ft (11.4 m) below the nominal grade elevation of 10 ft (3.0 m), with the top of the walls at elevation 11.5 ft (3.5 m) and the top of the basemat at elevation -22.5 ft (-6.9 m).

The UHS Makeup Water Intake Structure is a reinforced concrete structure 75 ft (22.9 m) long by 60 ft (18.3 m) wide by 54 ft (16.5 m) high, including a 5 ft (1.5 m) thick basemat that is integrally connected with the Forebay basemat. The structure consists of a below-grade water basin 59 ft (18.0 m) long by 60 ft (18.3 m) wide by 39 ft (11.9 m) deep situated approximately 37.5 ft (11.4 m) below the nominal grade elevation of 10 ft (3.0 m) and an above-grade pump house structure situated partially above the water basin and partially over structural fill.

The three main elevations of the UHS Makeup Water Intake Structure are:

- Elevation -22.5 ft (-6.9 m): Bottom of the water basin and top of the basemat. There are four independent pump bays in the water basin, separated by reinforced concrete walls.
- Elevation 11.5 ft (3.5 m): Top of the operating deck and pump house floor, which includes four make-up water pump rooms and four traveling screen rooms separated by reinforced concrete walls. The pump rooms are water-tight to protect against hurricane floods.
- Elevation 26.5 ft (8.1 m): Top of the nominally 2 ft (0.6 m) thick, reinforced concrete roof slab. Water-tight equipment hatches are provided for equipment maintenance.

Functional components within the water basin include UHS Makeup Water pumps, UHS Makeup Water Screen Wash Pumps, intake bar screens and traveling screens to preclude debris intake, and stop logs provision to facilitate maintenance.

Exterior walls for the pump house are 2 ft (0.6 m) thick, to withstand tornado missile impact and the wave pressures of the Probable Maximum Hurricane (PMH) extreme environmental event and the Standard Project Hurricane (SPH) severe environmental event. Interior walls that are subject only to minor lateral loads are one ft (0.3 m) thick. The divider and exterior walls of the basin of the UHS Makeup Water Intake Structure are all 4 ft (1.2 m) thick. A 2.5 ft (0.8 m) thick, inclined partial-height wall faces the Forebay.

The Seismic Category I UHS Electrical Building is 33 ft (10 m) wide by 74 ft (22 m) long by 21 ft (6.4 m) deep, including a 5 ft (1.5 m) thick basemat. It is constructed entirely of reinforced concrete and contains four electrical rooms, each of which houses: a transformer, a motor control center, and associated cooling equipment. To mitigate the effects of the PMH wave pressures, the UHS Electrical Building is almost entirely embedded in the surrounding soil, with its roof situated at Elevation 10.5 ft (3.2 m), or 6 in (15 cm) above grade, and top of the basemat situated at Elevation -5.5 ft (-1.7 m). A 10 ft (3.0 m) high reinforced concrete enclosure above the roof protects the access stairs.

The UHS Electrical Building has 2 ft (0.6 m) thick exterior walls, interior walls, and roof slab. The walls are sized to provide sufficient dead load to oppose the significant buoyant forces during the PMH and SPH events. The roof slab is sized and reinforced to protect against external hazards (e.g., tornado missile impact). Water-tight equipment hatches are provided for equipment maintenance.}

3.8.4.2 Applicable Codes, Standards, and Specifications

No departures or supplements.

3.8.4.3 Loads and Load Combinations

The U.S. EPR FSAR includes the following COL Item in Section 3.8.4.3:

> A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard design envelope for other Seismic Category I structures, or perform additional analyses to verify structural adequacy.

This COL Item is addressed as follows:

{Table 2.0-1 provides a comparison of CCNPP Unit 3 site parameters to the parameters defining the basis of the U.S. EPR FSAR design loads. Site-specific load parameters, except the site-specific safe shutdown earthquake (SSE), are bounded by the parameters defined for the U.S. EPR FSAR.

As described in Section 3.7.2.5.2, in-structure response spectra (ISRS) for the EPGB and ESWB based on the site-specific SSE exceed the ISRS based on the certified seismic design response spectra (CSDRS) for frequencies approximately below 0.3 Hz. The maximum spectral acceleration below 0.3 Hz is 0.07g. The structural reconciliation of EPGB and ESWB confirms that the structures as designed in U.S. EPR FSAR are adequate for site-specific SSE loading.

Design loads and load combinations for site-specific Seismic Category I structures are addressed in Section 3.8.4.3.1 and 3.8.4.3.2, respectively.}

3.8.4.3.1 Design Loads

{Design loads defined in the U.S. EPR FSAR Section 3.8.4.3.1 are applicable for the design of site-specific Seismic Category I structures, with the following exceptions:

- Live loads (L) — Design live load due to rain, snow and ice is based on the normal and extreme winter precipitation events described in Section 2.3.1.2.2.12.
- Thermal loads (To) — Thermal loads during normal operation are negligible based on site-specific temperature parameters (i.e., thermal gradient and uniform temperature change) and the guidelines provided in Section 1.3 of ACI 349.1R-07 (ACI, 2007).
- Soil loads and lateral earth pressure (H) — Static lateral soil pressure is calculated based on site-specific soil parameters and groundwater elevation. Design unit weight for the structural fill (95% Modified Proctor) used in the intake area is as follows:
  - Moist unit weight: 149 pcf

- Saturated unit weight: 153 pcf

Lateral earth pressure coefficients are defined in Table 2.5-58. A coefficient of 0.5 is used conservatively for the structural fill for at-rest condition. The groundwater table in the intake area is at about Elevation 3 ft. A normal surcharge load of 500 psf minimum is considered for calculating the lateral earth pressures. Lateral pressures due to compaction associated with structural fill are also considered.

- Safe shutdown earthquake (E') —
  - Site-specific SSE is defined in Section 3.7.1.1.1.1, which has a peak ground acceleration of 0.15 g, as shown in Figure 3.7-1.
  - Dynamic soil pressure: Effects of dynamic soil pressure on the intake structures are captured by the SSI analysis described in Section 3.7.2.4.
- Abnormal loads — Abnormal loads generated by a postulated high-energy pipe break accident are not applicable, since such pipes are not present in the subject structures.
- Operating basis earthquake (OBE) — OBE is defined in Section 3.7.1.1.1.1 and shown in Figure 3.7-6, which is essentially one-third of the site-specific SSE. As such, OBE loads are not explicitly considered for the design of the site-specific Seismic Category I structures.

Additional design loads for site-specific Seismic Category I structures include the severe and extreme environmental loads associated with the postulated standard project hurricane (SPH) and probable maximum hurricane (PMH) events, respectively.

The hurricane wave pressure on the exterior walls of the UHS Makeup Water Intake Structure and the UHS Electrical Building is obtained using the methodology presented in the Coastal Engineering Manual (USACE, 2006). The total wave pressure is equal to the sum of hydrostatic pressure and hydrodynamic wave pressure. The hydrostatic pressure is calculated based on the storm surge still water level of 14.1 ft (4.3 m) and 21.7 ft (6.6 m) NGVD 29 for SPH and PMH, respectively. The hydrodynamic wave pressure is calculated based on the 0.15% exceedance Rayleigh distributed wave height for the east and north exterior walls by plant reference direction. The wave pressures on the north and west walls are calculated based on the breaking wave heights corresponding to the still water depths. For structures that remain submerged under the storm surge still water level during a SPH or a PMH event (i.e., roof of the UHS Electrical Building, Forebay walls, and operating deck of the UHS Makeup Water Intake Structure), the dynamic wave pressure is calculated based on the stream function wave theory (Dean, 1974) for breaking wave conditions for wave steepness that provide the maximum dynamic pressures on these structures.

Concurrent hurricane wind speeds based on the 3 second wind gust at 32.8 ft (10 m) high are 110 mph (177 km/hr) and 195 mph (314 km/hr) for SPH and PMH, respectively. Conservatively, the concurrent hurricane wind pressure for design of site-specific Seismic Category I structures is based on the U.S. EPR standard design wind speeds of 145 mph

(233 km/hr) and 230 mph (370 km/hr), for SPH and PMH respectively, utilizing the procedures presented in Chapter 6 of ASCE 7-05 (ASCE, 2005).

Due to much higher grade elevation, structures in the powerblock area are not affected by the wave pressure associated with the postulated hurricanes. Concurrent hurricane wind loads are enveloped by the wind and tornado wind loads presented in the U.S. EPR FSAR Sections 3.3.1 and 3.3.2, respectively.

In addition, both the UHS Makeup Water Intake Structure and UHS Electrical Building are designed to withstand a peak positive incident overpressure (due to postulated explosions) of at least 1 psi without loss of function based on the guidance in RG 1.91, Rev. 1 (NRC, 1978a).}

3.8.4.3.2 Loading Combinations

{The following additional factored load combinations apply to the reinforced concrete design of the Forebay, UHS Makeup Water Intake Structure, UHS Electrical Building, and Seismic Category I buried electrical bank and piping in the intake area:

- Severe Environment SPH:

  $$U = 1.4\,(D + F) + 1.7\,(L + H + R_o + SPH)$$

- Extreme Environment PMH:

  $$U = D + F + L + H + R_o + PMH$$

These two load combinations are in addition to the load combinations specified in U.S. EPR FSAR Section 3.8.4.3.2.

For all the load combinations, according to ACI 349-01 (ACI, 2001a), if any load reduces the effects of other loads, the corresponding load factor is taken as 0.9 if that load is always present or occurs simultaneously with the other loads. Otherwise, the factor for that load is taken as zero.}

3.8.4.4 Design and Analysis Procedures

No departures or supplements.

3.8.4.4.1 General Procedures Applicable to Other Seismic Category I Structures

No departures or supplements.

3.8.4.4.2 Reactor Shield Building and Annulus, Fuel Building, and Safeguard Buildings – NI Common Basemat Structure

No departures or supplements.

3.8.4.4.3 Emergency Power Generating Buildings

No departures or supplements.

3.8.4.4.4 Essential Service Water Buildings

No departures or supplements.

3.8.4.4.5 Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts

The U.S. EPR FSAR includes the following COL Items in Section 3.8.4.4.5:

> A COL applicant that references the U.S. EPR design certification will describe the design and analysis procedures used for buried conduit and duct banks, and buried pipe and pipe ducts.
>
> A COL applicant that references the U.S. EPR design certification will use results from site-specific investigations to determine the routing of buried pipe and pipe ducts.
>
> A COL applicant that references the U.S. EPR design certification will perform geotechnical engineering analyses to determine if the surface load will cause lateral or vertical displacement of bearing soil for the buried pipe and pipe ducts and consider the effect of wide or extra heavy loads.

The COL Items identified above are addressed as follows:

{The analysis and design of Seismic Category I, buried electrical duct banks, buried Essential Service Water pipes, buried UHS Makeup Water Pipes, and buried CCNPP Unit 3 Intake Pipes (hereafter in this section referred to as buried duct banks and buried pipes) for all the imposed loads, follows the procedures outlined in U.S. EPR FSAR Section 3.8.4.4.5. The analysis and design of the buried pipes also follow the procedures described in Section 3.10 of the AREVA NP Topical Report ANP-10264NP-A (AREVA, 2008).

The design of buried duct banks and buried pipes demonstrates sufficient strength to accommodate:

- Strains imposed by seismic ground motion.
- Static surface surcharge loads due to vehicular loads (AASHTO HS-20 (AASHTO, 2002) truck loading, minimum, or other vehicular loads, including during construction) on designated haul routes.
- Static surface surcharge loads during construction activities, e.g., for equipment laydown or material laydown.
- Tornado missiles and, within their zone of influence, turbine generated missiles.
- Groundwater effects.

Terrain topography and the results from the CCNPP Unit 3 geotechnical site investigation will be used as design input to confirm the routing of buried pipe and duct banks reflected in Figures 3.8-1 through Figure 3.8-4.

The seismic design of buried duct banks and buried pipe is discussed in Section 3.7.3. Other loads are addressed in this section, but are combined with seismic effects of the aforementioned section.

Soil overburden pressures on buried duct banks and buried pipes typically do not induce significant bending or shear effects, because the soil cover and elastic support below the buried duct banks and buried pipes are considered effective and uniform over the entire length of the buried duct bank and buried pipe. When this is not the case, vertical soil overburden pressure is determined by the Boussinesq method.

Transverse stirrups used to reinforce the concrete duct banks are open ended to mitigate magnetic effects on the electrical conduits. Distribution of transverse and longitudinal steel reinforcement is sufficient to maintain the structural integrity of the electrical duct bank, for all imposed loads, in accordance with ACI 349-01 (ACI, 2001a) (with supplemental guidance of Regulatory Guide 1.142 (NRC, 2001)).

As noted in Section 3.8.4.1.9, buried pipes are located such that the top surface of the pipe is below the site-specific frost depth, with additional depth used to mitigate the effects of surcharge loads and tornado or turbine generated missiles. In lieu of depressing the pipes in the soil beyond that required for frost protection, i.e., to obviate the risk of tornado or turbine generated missile impacts, permanent protective steel plates, located at grade, may be designed.

Bending stresses in buried pipe due to surcharge loading are determined via manual calculations, treating the flexible pipe as a beam on an elastic foundation. Resulting stresses are combined with operational stresses, as appropriate.}

3.8.4.4.6 Design Report

{Design reports for the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are presented in Appendix 3E.4.}

3.8.4.4.7 {Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building

This section is added as a supplement to U.S. EPR FSAR Section 3.8.4.4.

The Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are reinforced concrete shear wall structures. Vertical loads are transferred to the foundation basemat through the reinforced concrete walls before being transferred to the supporting soil through bearing pressure. Lateral loads, including those that are seismically induced, are transferred to the supporting soil by the foundation basemats and below-grade walls through friction, adhesion, and passive soil pressure, if necessary.

A finite element (FE) model was created for the Seismic Category I Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building and Seismic Category II CWS Makeup

Water Intake Structure, using GT STRUDL (version 29.1). The CWS Makeup Water Intake Structure is included in the FE model since it is integrally connected to the Forebay, shown in Figure 9.2-4. Since the CWS Makeup Water Intake Structure, Forebay, and UHS Makeup Water Intake Structure share a common basemat, they are also known as the Common Basemat Intake Structures (or CBIS).

GT STRUDL is a commercial structural engineering computer program developed by Georgia Tech. QA and QC requirements for safety-related structures are documented in the vendor's validation and verification manuals. The program is accepted for use in accordance with Bechtel's engineering department and QA procedures. The program is in compliance with the requirements of ASME NQA-1-1994 (ASME, 1994). The GT STRUDL FE model is converted to a SASSI model using Bechtel computer code SASSI 2000, version 3.1, to perform soil-structure interaction (SSI) analysis. SSI analysis is discussed in Section 3.7.2.

The GT STRUDL FE model is also used to conduct static analysis under non-seismic loads to compute the structural responses, generate results for the design of reinforced concrete structural elements, and perform static stability and bearing pressure evaluations. The finite element analysis results from the SSI analysis and the static analysis are combined to determine the reinforced concrete design forces and moments under seismic load cases.

The FE model is described in detail in Section 3.7.2.3. Figure 3.7-26 depicts the FE model for the CBIS and the UHS Electrical Building. Note that only one half of the structures are modeled since the structures are symmetric about their North-South (N-S) centerline. Depending on the symmetry of the applied loads, either symmetric or anti-symmetric boundary conditions (defined in Table 3.7-9) are used for the nodes in the plane of symmetry. For applied loads that are symmetric about the N-S centerline, results are calculated using the symmetric boundary conditions. For applied loads that are anti-symmetric about the N-S centerline, results are calculated using the anti-symmetric boundary conditions.

For the static analysis, the soil medium below the foundation basemat is represented by soil spring elements. The modulus of subgrade reaction for the soil spring elements is based on the site-specific soil properties presented in Section 2.5.4. The modulus of subgrade reaction is calculated using an iterative process until soil pressures at the soil-structure interface converge.

Since only half of the structures are modeled, reactions on the full basemat are determined using the following equations:

Symmetric loads	Anti-Symmetric loads
R'x = Rx	R'x = -Rx
R'y = Ry	R'y = -Ry
R'z = -Rz	R'z = Rz

where Rx, Ry, and Rz are the reactions in GT STRUDL global coordinate system calculated from FE analysis and R'x, R'y, and R'z are the corresponding reactions for the remaining half of the structure not included in the FE model.

Effects of the following loads are calculated from the static analysis: dead loads, live loads (including snow loads), hydrostatic loads, lateral earth pressure loads (including groundwater effects), wind loads, tornado loads (including wind pressure and differential pressure effects), SPH and PMH loads (including hydrostatic pressure, buoyancy, wave pressure, and concurrent wind pressure effects).

During maintenance of the UHS Makeup Water Intake Structure, when stop logs are installed, interior or exterior below-grade cells may be empty. The exterior embedded walls, with the empty adjacent cell, are subject to lateral soil pressure, surcharge and hydrostatic pressure from a normal groundwater level of +3 ft (0.9 m) NVGD 29. This postulated maintenance condition is considered in the FE model for designing the side walls of the UHS Makeup Water Intake Structure.

Seismic induced hydrodynamic loads associated with the water contained in the CBIS are calculated according to the provisions of ACI 350.3-06 (ACI, 2006). Effects of the impulsive component of the hydrodynamic loads are calculated in the SSI analysis by including the corresponding water mass in the SASSI model. Effects of the convective component are negligible for both global and local responses, except for the local response of the front wall of the UHS Makeup Water Intake Structure facing the Forebay, which will be considered during detailed design.

As described earlier, finite element forces and moments from the aforementioned SSI and static analyses are combined to generate design forces and moments for load combinations involving seismic effects, in accordance with Section 3.8.4.3.2. Seismic moments and forces for a particular earthquake direction are computed by enveloping the forces and moments for that direction for all soil profiles (i.e., UB, BE, LB described in Section 3.7.1.3.3).

Enveloped SASSI results from the three components of earthquake motions are combined using the 100-40-40 percent rule, as described in Section 3.7.2.6. The design forces and moments from seismic and non-seismic load combinations are used to design reinforced concrete shear walls and slabs according to the provisions of ACI 349-01 (ACI, 2001a) (with supplemental guidance of Regulatory Guide 1.142 (NRC, 2001)). Results of the reinforced concrete design are provided in Appendix 3E Section 3E.4.5.

The evaluation of slabs and walls for external hazards (e.g., tornado generated missiles) is performed by local analyses, following the procedure outlined in U.S. EPR FSAR Section 3.8.4.4.1. Procedures for stability evaluation and bearing pressure calculation are discussed in Section 3.8.5.4.6.}

3.8.4.5 Structural Acceptance Criteria

The U.S. EPR FSAR includes the following COL Item in Section 3.8.4.5:

> A COL applicant that references the U.S. EPR design certification will confirm that site-specific conditions for Seismic Category I buried conduit, electrical duct banks, pipe, and pipe ducts satisfy the criteria specified in Section 3.8.4.4.5 and those specified in AREVA NP Inc., U.S. Piping Analysis and Support Design Topical Report.

This COL Item is addressed as follows:

Design of all safety-related, Seismic Category I buried electrical duct banks and pipe meet the requirements specified in U.S. EPR FSAR Section 3.8.4.4.5 and the Areva NP Topical Report ANP-10264NP-A (AREVA, 2008).

Acceptance criteria for the buried electrical duct banks are in accordance with IEEE 628-2001(R2006) (IEEE, 2001), ASCE 4-98 (ASCE, 2000) and ACI 349-01 (ACI, 2001a), with supplemental guidance of Regulatory Guide 1.142 (NRC, 2001). The use of ACI 349-01, in lieu of ACI 349-97 (ACI, 1997) as invoked in Subsection 4.9.4.15 of IEEE 628-2001 (R2006), is to provide a consistent design basis with all other Seismic Category I structures.

{Acceptance criteria for the buried UHS Makeup Water Pipes and CCNPP Unit 3 Intake Pipes are identical to that stated above. Member stresses are maintained lower than allowable stresses. When allowable stresses are exceeded, joints are added as required to increase flexibility and hence, to mitigate member stresses.

Acceptance criteria for the reinforced concrete design of site-specific Seismic Category I Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are identical to those described in the U.S. EPR FSAR Section 3.8.4.5.}

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

No departures or supplements.

3.8.4.6.1 Materials

{As discussed in Section 2.5.4.2.5.2, all natural soils at the site are considered aggressive to concrete. However, structures and buried duct banks and pipes will be surrounded and supported by non-aggressive structural fill obtained from off-site borrow sources. Hence, the durability requirements of below-grade concrete walls and buried duct banks and pipes are based on the chemical properties of groundwater.

There are two hydrogeologic units of groundwater affecting the CCNPP Unit 3 structures and buried utilities - the Surficial aquifer, which is present in the powerblock area only, and the upper Chesapeake unit, which underlies both the intake and the powerblock areas. Observed groundwater chemical properties (pH, sulfates and chlorides) for the Surficial aquifer and the Upper Chesapeake unit are provided in Table 3.8-5.

Comparing the observed pH, sulfate, and chloride values with the SRP 3.8.4 (NRC, 2007) acceptance criteria for aggressive groundwater, i.e., pH < 5.5, chlorides > 500 parts per million (ppm), and/or sulfates > 1500 ppm, groundwater from the Surficial aquifer in the powerblock area is considered aggressive due to its low pH-value. Groundwater in the intake area is considered non-aggressive.

As stated in Section 2.4.12.5, the post-development groundwater elevation in the powerblock area is at about 30 ft (9.1 m) below the finished site grade level of 85 ft (25.9 m). The NI common basemat structures are embedded approximately 40 ft (12.2 m) below the finished grade. Therefore, the lower portions of the NI common basemat structures are submerged in the low-pH groundwater from the Surficial aquifer. Other Seismic Category I structures in the powerblock area, i.e., the EPGBs and ESWBs, are located above the post-development groundwater level and are not affected by the low-pH groundwater.

A waterproofing system is provided to protect the reinforced concrete NI common basemat structures from the corrosive effects of low-pH groundwater. As illustrated in Figure 3.8-6, the waterproofing system consists of a primary geomembrane envelope located under the NI common basemat and mud mat between two sand layers and attached to the below-grade walls, extending up to Elevation 57'-0" (17.4 m) NGVD 29, or about 2 ft (0.6 m) above the highest projected post-development groundwater level. Secondary waterproofing starts at the bottom of the below-grade walls, continues above the groundwater level and terminates at about 1 ft (0.3 m) above the finished grade level. A groundwater monitoring system (consisting of risers and drain sumps) is provided inside the geomembrane envelope within the sand layer to monitor and pump out any water that may leak through the primary geomembrane. A vertical drainage layer is placed between the primary and secondary waterproofing membranes to facilitate the flow of any leaked groundwater down to the sumps.

A majority of the buried electrical duct banks are located above the post-development groundwater level in the powerblock area and are not affected by the low-pH groundwater. For the duct banks in the utility corridor that may be exposed to the low-pH groundwater, liquid-applied or geomembrane waterproofing is applied for protection against prolonged exposure to the groundwater. Protective measures for buried pipe include protective wrapping and/or coatings that are acid-resistant.

Since the groundwater is non-aggressive in the intake area, waterproofing is not needed for the protection of concrete structures or duct banks. However, due to a high groundwater table, a waterproofing membrane is provided for the UHS Electrical Building to protect the enclosed electrical equipment from water intrusion.

As noted in Table 3.8-5, the maximum observed sulfate concentration in the groundwater is 365 ppm. According to ACI 349-01 (ACI, 2001a) Table 4.3.1, this concentration is considered a moderate exposure (also identified as "Class 1 Exposure" in ACI 201.2R-01 (ACI, 2001b)) and requires the use of ASTM C150 (ASTM, 2009) Type II or equivalent cement, a maximum water-cementitious materials ratio of 0.5, and a minimum concrete compressive strength of 4000 psi.

Additionally, for concrete structures subject to the brackish water from the Chesapeake Bay, Table 4.2.2 of ACI 349-01 (ACI, 2001a) requires the use of a maximum water-cementitious materials ratio of 0.4 and a minimum specified compressive strength of 5000 psi.

Based on aforementioned requirements, concrete mixtures for Seismic Category I Forebay, UHS Makeup Water Intake Structure, UHS Electrical Building, and buried utilities (i.e., buried concrete electrical duct banks and pipes) will have a maximum water-cementitious materials ratio of 0.4 and a minimum specified compressive strength of 5000 psi. A maximum water-cementitious materials ratio of 0.4 is also specified for the Essential Service Water Buildings as they can also be exposed to the brackish water. Concrete mixtures for other Seismic Category I structures will have a maximum water-cementitious materials ratio of 0.45.

For improved resistance to sulfate attack and chloride ion penetration, about 20-25% of the total weight of the cementitious materials in all concrete mixtures will be replaced with fly ash (conforming to ASTM C618 (ASTM, 2005) Class F) to limit temperature gain, thus reducing peak hydration temperature and permeability of the concrete.}

3.8.4.6.2 Quality Control

No departures or supplements.

3.8.4.6.3 Special Construction Techniques

{Special construction techniques are not expected to be used for the Seismic Category I Emergency Power Generating Buildings, Essential Service Water Buildings, Forebay, UHS Makeup Water Intake Structure, UHS Electrical Building and buried utilities.}

3.8.4.7 Testing and Inservice Inspection Requirements

{As discussed in Section 3.8.4.6.1, although the CCNPP Unit 3 in-situ soil is aggressive to concrete, it will be replaced by non-aggressive structural fill under and around the structures and buried duct banks and buried pipes. In addition, Seismic Category I structures and buried utilities in the powerblock area are protected by waterproofing or coating, if submerged in the low-pH groundwater from the Surficial aquifer. As a result, the structures and buried utilities are not directly exposed to the in-situ soil or the low-pH groundwater.

For normally inaccessible below-grade concrete walls and foundations and buried utilities that are not exposed to low-pH groundwater, the inservice inspection program is limited to examination of the exposed portions of below-grade concrete walls and buried utilities for signs of degradation, when excavated for any reason. Exposed geomembrane and related waterproofing systems are also inspected during the excavation.

For the NI common basemat structures, in-service inspection utilizes a groundwater monitoring system consisting of risers and drain sumps. The risers and sumps will be subject to periodic monitoring to confirm that groundwater leaking through the geomembrane envelope is being effectively removed and is not ponded against the concrete structure. Such monitoring will:

- Occur at multiple locations in the monitoring system;
- Be performed on a frequency based on the leakage rate through the primary geomembrane. The leakage rate will be determined by monitoring water levels in the risers and drain sumps. Initially the monitoring frequency is expected to be high until the performance of the geomembrane is established. As the operation proceeds, the monitoring interval will be expanded, possibly to once per cycle;;
- Utilize manual techniques or electronic water level sensors.

The buried duct banks have shallow embedment depth. Therefore, the condition of the buried concrete duct banks in the utility corridor that may be exposed to low-pH groundwater of the Surficial aquifer will be monitored by excavating the surrounding soil. The frequency of this monitoring will be determined based on the groundwater level and pH values recorded by the groundwater monitoring program described in Section 3.8.5.7.

Groundwater levels throughout the powerblock area will also be monitored to confirm that no other below-grade concrete requires dewatering provisions to protect it from prolonged exposure to the low-pH groundwater from the Surficial aquifer. The groundwater chemical properties are monitored through the monitoring program described in Section 3.8.5.7.

The in-service inspection program and performance monitoring will be designed and conducted in conformance with the requirements of 10 CFR 50.65 (CFR, 2008) and Regulatory Guide 1.160 (NRC, 1997). The in-service inspection program for the UHS Makeup Water Intake Structure and the Forebay are developed and conducted in accordance with Regulatory Guide 1.127 (NRC, 1978b). The in-service program includes below-grade walls and buried utilities addressed in this section, as well as foundations addressed in Section 3.8.5.}

3.8.5 FOUNDATIONS

3.8.5.1 Description of the Foundations

The U.S. EPR FSAR includes the following COL Item in Section 3.8.5.1:

> A COL applicant that references the U.S. EPR design certification will describe site-specific foundations for Seismic Category I structures that are not described in this section.

This COL Item is addressed as follows:

{The foundations for the site-specific Seismic Category I Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building are discussed in Section 3.8.5.1.4.}

3.8.5.1.1 Nuclear Island Common Basemat Structure Foundation Basemat

No departures or supplements.

3.8.5.1.2 Emergency Power Generating Buildings Foundation Basemats

No departures or supplements.

3.8.5.1.3 Essential Service Water Buildings Foundation Basemats

No departures or supplements.

3.8.5.1.4 {Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building Basemats

This section is added as a supplement to the U. S. EPR FSAR.

Plans, sections and details for the Seismic Category I Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are provided in Figures 9.2-4, 9.2-5 and 9.2-6. A general description of the structures, including descriptions of all functional levels, is provided in Section 3.8.4.1.11. Figure 3.8-1 shows the position of the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building relative to the NI.

As shown in Figure 9.2-4, Seismic Category II CWS Makeup Water Intake Structure and Seismic Category I Forebay and UHS Makeup Water Intake Structure share a 5 ft (1.5 m) thick common basemat, with its top elevation at -22.5 ft (-6.9 m). The UHS Electrical Building sits on a separate 5 ft (1.5 m) thick basemat, with its top elevation at -5.5 ft (-1.7 m).

The reinforced concrete basemat for the Forebay is 109 ft (33.2 m) long by 89 ft (27.1 m) wide. The reinforced concrete basemat for the UHS Makeup Water Intake Structure is 59 ft (18.0 m) long by 60 ft (18.3 m) wide, while that for the UHS Electrical Building is 33 ft (10.1 m) long by 74 ft (22.6 m) wide. Concrete walls bearing on the foundation basemats of Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are described in Section 3.8.4.1.11 and shown on Figures 3E.4-1 and 3E.4-2.

Lateral loads, including those that are seismically induced, are transferred to the supporting soil by the foundation basemats and below-grade walls through friction, adhesion, and passive soil pressure, if necessary. Vertical forces from the super structures are transferred to the foundation basemat through the bearing walls, before being transferred to the supporting soil through bearing pressure.}

3.8.5.2 Applicable Codes, Standards, and Specifications

No departures or supplements.

3.8.5.3 Loads and Load Combinations

{Structural loads and load combinations for reinforced concrete basemat design of site-specific Seismic Category I structures are defined in Sections 3.8.4.3.1 and 3.8.4.3.2.}

Load combinations for stability evaluation, including sliding, overturning, and floatation, are described in U.S. EPR FSAR Section 3.8.5.3. Additional stability load combinations for sliding and overturning evaluations include:

- D + H + SPH
- D + H + PMH

These load combinations are analogous to the stability load combinations for wind and tornado loading.

Load combinations for bearing pressure evaluation are as follows:

Service loads

- D + L + F

Severe environmental loads

- D + L + F + W
- D + L + F + SPH

Extreme environmental loads

- D + L + F + Wt
- D + L + F + E'
- D + L + F + PMH

Buoyancy due to groundwater reduces bearing pressure and is neglected from above load combinations.}

3.8.5.4 Design and Analysis Procedures

No departures or supplements.

3.8.5.4.1 General Procedures Applicable to Seismic Category I Foundations

No departures or supplements.

3.8.5.4.2 Nuclear Island Common Basemat Structure Foundation Basemat

No departures or supplements.

3.8.5.4.3 Emergency Power Generating Buildings Foundation Basemats

No departures or supplements.

3.8.5.4.4 Essential Service Water Buildings Foundation Basemats

No departures or supplements.

3.8.5.4.5 Design Report

{Design reports for the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building basemats are presented in Appendix 3E.4.}

3.8.5.4.6 {Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building Basemats

This section is added as a supplement to U.S. EPR FSAR Section 3.8.5.4.

As shown in Figure 3.7-26, the foundation basemats are part of the finite element model used for the analysis and design of the Seismic Category I Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building. The finite element mesh of the basemats is shown in Figure 3.8-5. Note that only half of the basemat is modeled because of symmetry. Analysis and critical section design procedures for these structures are presented in Section 3.8.4.4.7.

To ensure the stability of the structures during various design basis events, the Common Basemat Intake Structures (CBIS) and the UHS Electrical Building are checked for sliding, overturning, and floatation using the stability load combinations described in Section 3.8.5.3.

Static and dynamic bearing pressures are calculated and compared with the bearing capacities defined in Table 2.5-65. Both average pressure and pressure at the basemat corner points (toe pressure) are calculated and checked.

Results from the SASSI analysis are used to calculate sliding forces and overturning moments for seismic loads, as described in Section 3.7.2.14.2. The loads contributing to the structural mass in the SSI analysis are used to calculate the resistance to sliding and overturning. These loads include 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.

For the non-seismic loads, basemat reactions from GT STRUDL analysis are used to calculate sliding forces and overturning moments. The dead load used to calculate the resistance to sliding and overturning includes the self weight of the structures, permanent equipment and water inside structures during the normal operation, SPH and PMH conditions.

Floatation is checked under normal operation, SPH, and PMH conditions, including the draw-down condition during a PMH event, with the water inside the CBIS at the minimum design level of -8 ft (-2.4 m). Resistance to floatation is provided by dead load.

Sliding is checked at various sliding interfaces below the foundation basemats. The CBIS sits on top of a mud mat, which is placed directly on the in-situ soil stratum IIc (Chesapeake clay/silt). Therefore, resistance to sliding is provided by friction between the basemat and the mud mat and friction and adhesion between the mud mat and soil stratum IIc. Friction (traction) between the below-grade walls and structural fill is also utilized for SSE loads. Passive soil pressure is not utilized for the stability of the CBIS.

The UHS Electrical Building is surrounded by structural fill and sits on top of a mud mat. To prevent water intrusion, waterproofing membrane is utilized between two sand layers below the mud mat, with a configuration similar to that shown in Figure 3.8-6. The bottom sand layer is underlain by structural fill. Resistance to sliding is provided by friction across the various interfaces. In addition, passive soil pressure is utilized to prevent the building from sliding under SSE and PMH loads.

The static coefficients of friction for various sliding interfaces are presented in Table 3.8-1. When passive resistance is utilized for the UHS Electrical Building, the static coefficient of friction is conservatively reduced by 25% to estimate dynamic coefficient of friction based on the triaxial laboratory test results performed on the soils.

Frictional resistance is reduced by the effects of any upward forces, such as upward seismic forces and buoyancy. Overturning resistance is reduced by buoyancy.

The factors of safety from aforementioned stability evaluations are compared with the minimum required factors of safety specified in U.S. EPR FSAR Table 3.8-11. The minimum required factors of safety for sliding and overturning associated with SPH and PMH are the same as those for wind and tornado, respectively. The minimum required factor of safety for floatation, including SPH and PMH conditions, is 1.1.

Results of the stability and bearing pressure evaluations are presented in Section 3.8.5.5.4.}

3.8.5.5 Structural Acceptance Criteria

The U.S. EPR FSAR includes the following COL Item in Section 3.8.5.5:

> A COL applicant that references the U.S. EPR design certification will evaluate site-specific methods for shear transfer between the foundation basemats and underlying soil for soil parameters that are not within the envelope specified in Section 2.5.4.2.

This COL Item is addressed as follows:

{For the Nuclear Island (NI) common basemat structures, Emergency Power Generating Buildings (EPGBs), and Essential Service Water Building (ESWBs), U.S. EPR FSAR Section 2.5.4.2 specifies a coefficient of friction of 0.7 beneath their basemats. As identified in Table 2.5-58, the geotechnical site investigation for CCNPP Unit 3 indicates a coefficient of friction between 0.35 and 0.45 for underlying soil layers. This represents a departure from the friction coefficient of 0.7 specified in the U.S. EPR FSAR.

A site-specific sliding evaluation for SSE loads is performed to confirm the sliding stability of NI common basemat structures, EPGBs, and ESWBs. These structures are located in the powerblock area, which will be excavated and backfilled. Mud mats are used under the basemat of each structure to facilitate construction. As described in Section 3.8.4.6.1, a waterproofing system is used to protect the NI common basemat structures from the low-pH groundwater, as illustrated in Figure 3.8-6. The potential sliding interfaces down to the natural soils under the NI common basemat structures are:

- Basemat — mud mat
- Mud mat — sand
- Sand — waterproofing membrane
- Sand — structural fill
- Structural fill — soil stratum IIb

No waterproofing is used for the EPGBs and ESWBs because they are located above the post-development groundwater table. Therefore, the potential sliding interfaces under the EPGBs and ESWBs are:

- Basemat—mud mat
- Mud mat—structural fill
- Structural fill — soil stratum IIb

Frictional parameters at the various sliding interfaces are presented in Table 3.8-1. Based on these frictional parameters, factors of safety against sliding and overturning associated with the site-specific SSE loads are presented in Table 3.8-4 for the NI common basemat structures, EPGBs, and ESWBs. The minimum required factor of safety of 1.1 is achieved for all the buildings. Note that passive soil pressure is not utilized for the sliding evaluation.}

3.8.5.5.1 Nuclear Island Common Basemat Structure Foundation Basemat

{The following departure is taken from U.S. EPR FSAR Section 3.8.5.5.1.

The standard design of Seismic Category I foundations for the U.S. EPR is based on a maximum differential settlement of ½ inch in 50 ft in any direction across the foundation. These standard design values are specified in the U.S. EPR FSAR Sections 2.5.4.10.2 and 3.8.5.5.1, and tabulated in U.S. EPR FSAR Tier 1 Table 5.0-1. The expected site-specific values for settlement of the CCNPP Unit 3 NI Common basemat foundation are in the range of 1/600 (1 inch in 50 ft) to 1/1200 (½ inch in 50 ft) as stated in Section 2.5.4.

To account for the Calvert Cliffs site-specific expected differential settlement values, an evaluation of differential settlements up to 1 inch in 50 ft was performed. A static analysis was performed of the foundation structures assuming this site-specific differential settlement value. The static analysis was performed using the same finite element model developed by AREVA for the standard plant differential settlement criteria of ½ inch in 50 ft. The finite element model is analyzed using the QA verified software ANSYS V10.0 SP1.

The evaluation consisted of a static finite element analysis of the foundation structures which considered the effects of the higher expected displacement (tilt) on the foundation bearing pressures and basemat stress due to structural eccentricities resulting from a uniform rotation of the foundation mat along the axis of the NI Common basemat. The evaluation assumed no changes in the soil stiffness or increased flexure due to differential settlement consistent with the design analysis for the standard U.S. EPR design. The evaluation considered Soil Case SC15, from the U.S. EPR FSAR standard design, which represented the softest soil condition used in the U.S. EPR standard plant design and exhibits the largest differential displacements of the basemat.

The displacement is defined per length of the structure, 1 inch in 50 ft. The displacement of the NI common basemat is greatest along the North/South axis at the Fuel Building (FB) and least along this axis at Safeguard Building 2 and 3 (SB 2/3). Therefore, the NI model is rotated around the X-axis (West/East axis). The overall length of the NI basemat from the North end to the South end is approximately 344 ft (105 m). Since an initial settlement of 1 inch in 50 ft is considered, the NI structure has an initial displacement of approximately 7.0 inches (17.8 cm), or approximately 0.1 degrees.

Results from the evaluation indicate there is negligible difference in both the soil bearing pressures and the stresses in the concrete basemat structure when the NI is subjected to an initial settlement of 1 inch in 50 ft as compared to an initial settlement of ½ inch in 50 ft established in the U.S. EPR standard plant.

There is a negligible difference in both the bearing pressures and the stresses in the basemat when the NI is subjected to structural eccentricities associated with a 7 inch (17.8 cm) basemat differential displacement representing a settlement value of 1 inch in 50 ft. Therefore, the site-specific departure in differential settlement values is structurally acceptable.}

3.8.5.5.2 Emergency Power Generating Buildings Foundation Basemats

{The following departure is taken from U.S. EPR FSAR Section 3.8.5.5.2.

Section 2.5.4.10.2 of the U.S. EPR FSAR states that:

> "The design of Seismic Category I foundations for the U.S. EPR is based on a maximum differential settlement of ½ inch per 50 ft in any direction across the basemat."

The U.S. EPR FSAR maximum allowable differential settlement of ½ inch per 50 ft may also be expressed as a fraction, i.e., 1/1200. According to Section 2.5.4.10.2, the estimated site-specific differential settlement is 1/1166, which is about 3% higher than the allowable value described in the U.S. EPR FSAR.

A finite element analysis of the entire EPGB structure, including CCNPP Unit 3 site-specific soil springs, indicates the maximum differential settlement within the confines of the EPGB basemat is 1/2714, or substantially less than the allowable value of the U.S. EPR FSAR. The variation of the finite element analysis differential settlement (1/2714) with the estimated differential settlement value of 1/1166 is attributed to the conventional geotechnical treatment of the foundation as a flexible plate, a condition much more conservative than the actual 6 ft thick reinforced concrete basemat.

To verify the finite element analysis results, a manual calculation is performed for a selected beam strip (1 ft (0.3 m) wide by 6 ft (1.8 m) deep) of the EPGB basemat, plan view of which is shown in U.S. EPR FSAR Figure 3E.2-3. The beam strip is located at the centerline of the basemat and is perpendicular to the center reinforced concrete bearing wall. The selected two-span beam strip is 96 ft (29.3 m) long, with the aforementioned center wall and two parallel primary reinforced concrete bearing walls serving as pinned supports. Soil bearing pressures are applied to the beam strip and beam deflection is calculated. The calculation results confirm similar findings as the finite element analysis results, i.e., the maximum differential settlement of the EPGB basemat is substantially less than 1/1200.

To further evaluate the effects of the higher site-specific differential settlement, a finite element analysis of the entire EPGB is performed to evaluate the effect of a more conservative overall building tilt of L/550, where L is the least basemat dimension. For this analysis:

- Spring stiffnesses are adjusted until a tilt of L/550 is achieved.
- The elliptical distribution of soil springs is maintained.
- Soil spring stiffnesses along the centerline of the basemat (perpendicular to the direction of tilt) are retained.

- Adjustment is made to all other springs as a function of the distance from the basemat centerline.

The finite element analysis results show that increase in EPGB basemat design moment based on the more conservative differential settlement value of 1/550 (based on the overall tilt) is less than 3% of the U.S. EPR FSAR maximum design moment. Therefore, EPGB basemat is structurally adequate to resist the increased moments.}

3.8.5.5.3 Essential Service Water Buildings Foundation Basemats

{The following departure is taken from U.S. EPR FSAR Section 3.8.5.5.3.

U.S. EPR FSAR Section 2.5.4.10.2 states that:

> "The design of Seismic Category I foundations for the U.S. EPR is based on a maximum differential settlement of ½ inch per 50 ft in any direction across the basemat."

The U.S. EPR FSAR maximum allowable differential settlement of ½ inch per 50 ft may also be expressed as a fraction, i.e., 1/1200.

According to Section 2.5.4.10.2, the maximum site-specific differential settlement is 1/845, which exceeds the allowable value specified in the U.S. EPR FSAR.

A finite element analysis of the entire ESWB structure, including CCNPP Unit 3 site-specific soil springs, indicates the maximum differential settlement within the confines of the ESWB basemat is 1/1417, or less than the allowable value of the U.S. EPR FSAR. The variation of the finite element analysis differential settlement (1/1417) with the estimated differential settlement value of 1/845 is attributed to the conventional geotechnical treatment of the foundation as a flexible plate, a condition much more conservative than the actual 6 ft thick reinforced concrete basemat.

To verify the finite element analysis results, a manual calculation is performed for a selected beam strip (1 ft (0.3 m) wide by 6 ft (1.8 m) deep) of the ESWB basemat, plan view of which is shown in U.S. EPR FSAR Figure 3E.3-3. The beam strip is located at the centerline of the basemat and is perpendicular to the reinforced concrete bearing wall separating the two cooling towers. The selected two-span beam strip extends for the length of the two cooling towers, with the aforementioned divider wall and two parallel reinforced concrete bearing walls serving as pinned supports. Soil bearing pressures are applied to the beam strip and beam deflection is calculated. The calculation results confirm similar findings as the finite element analysis results, i.e., the maximum differential settlement of the ESWB basemat is less than 1/1200.

To further evaluate the effects of the higher site-specific differential settlement, a finite element analysis of the entire ESWB is performed to evaluate the effect of a more conservative overall building tilt of L/600, where L is the least basemat dimension. For this analysis:

- Spring stiffnesses are adjusted until a tilt of L/600 is achieved.

- The elliptical distribution of soil springs is maintained.
- Soil spring stiffnesses along the centerline of the basemat (perpendicular to the direction of tilt) are retained.
- Adjustment is made to all other springs as a function of the distance from the basemat centerline.

The finite element analysis results show that increase in the ESWB basemat design moments based on the more conservative differential settlement value of 1/600 (based on the overall tilt) is less than 5% of the U.S. EPR FSAR maximum design moments. So, the ESWB basemat is structurally adequate to resist the increased moments.}

3.8.5.5.4 {Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building Basemats

This section is added as a supplement to U.S. EPR FSAR Section 3.8.5.5.

Acceptance criteria for reinforced concrete design of basemat critical sections are described in Section 3.8.4.5.

Stability and bearing pressure of the CBIS and the UHS Electrical Building are evaluated following the procedures presented in Section 3.8.5.4.6. As reported in Table 3.8-2, factors of safety from various stability load combinations show that the minimum required values are achieved. Therefore, the CBIS and the UHS Electrical Building are stable under various design conditions.

For the UHS Electrical Building, approximately 30% of full passive resistance is utilized for the most critical loading condition (SSE) to achieve a minimum sliding factor of safety FOS = 1.1, resulting in an estimated movement of approximately ¼ inch. During detailed design, the utility connections to the UHS Electrical Building will be designed to accommodate this movement.

Maximum soil bearing pressures under the CBIS and the UHS Electrical Building foundations are provided in Table 3.8-3. The calculated maximum bearing pressures are smaller than the bearing capacities presented in Table 2.5-65 under both static and dynamic conditions.

Differential settlement across the CBIS and UHS Electrical Building is within the U.S. EPR FSAR differential settlement criterion of 1/1200.}

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

No departures or supplements.

3.8.5.6.1 Materials

The U.S. EPR FSAR includes the following COL Item in Section 3.8.5.6.1:

> A COL applicant that references the U.S. EPR design certification will evaluate and identify the need for the use of waterproofing membranes and epoxy coated rebar based on site-specific ground water conditions.

This COL Item is addressed as follows:

{The waterproofing membrane is used to eliminate the prolonged exposure of below grade concrete from the low pH groundwater of Surficial aquifer, as described in Section 3.8.4.6.1. Discussion of concrete mix design for improved resistance to sulfate attack and chloride ion penetration is also presented in Section 3.8.4.6.1. Epoxy coated rebar is not used.}

3.8.5.6.2 Quality Control

No departures or supplements.

3.8.5.6.3 Special Construction Techniques

{Special construction techniques are not expected to be used for the Emergency Power Generating Buildings, Essential Service Water Buildings, Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building.}

3.8.5.7 Testing and Inservice Inspection Requirements

The U.S. EPR FSAR includes the following COL Items in Section 3.8.5.7:

> A COL applicant that references the U.S. EPR design certification will identify if any site-specific settlement monitoring requirements for Seismic Category I foundations are required based on site-specific soil conditions.
>
> A COL applicant that references the U.S. EPR design certification will describe the program to examine inaccessible portions of below-grade concrete structures for degradation and monitoring of groundwater chemistry.

These COL Items are addressed as follows:

{Although settlement and differential settlement of foundations are not likely to affect the structures, systems, and components that make up the U.S. EPR standard plant due to the robust design of all Seismic Category I structures, a site-specific settlement monitoring program is required as a prudent measure of confirmation between expected or predicted settlement and actual field measured settlement values.

The settlement monitoring program employs conventional monitoring methods using standard surveying equipment and concrete embedded survey markers. Survey markers are embedded in the concrete structures during construction and located in conspicuous locations above grade for measurement purposes throughout the service life of the plant as necessary. Actual field settlement is determined by measuring the elevation of the marker relative to a reference elevation datum. The reference datum selected is located away from areas susceptible to vertical ground movement and loads. If field measured settlements are found to be trending greater than expected values, an evaluation will be conducted to ensure compliance with design basis requirements.

The settlement monitoring program shall satisfy the requirements for monitoring the effectiveness of maintenance specified in 10 CFR 50.65 (CFR, 2008) and Regulatory Guide 1.160 (NRC, 1997), as applicable to structures.

The CCNPP Unit 3 below-grade concrete degradation monitoring program is described in Section 3.8.4.7. This program calls for:

- Examination of exposed portions of below-grade concrete, including buried utilities, for signs of degradation when excavated for any reason; and
- Periodic monitoring of risers and drain sumps for the NI common basemat structures to ensure that the groundwater leaking through the geomembrane envelope, if any, is being effectively removed and is not ponding against the concrete structure.

As stated in Section 3.8.4.7, groundwater levels throughout the powerblock area will be monitored. The CCNPP Unit 3 groundwater monitoring program is established on the following bases:

- Recorded baseline concentrations and pH values of groundwater chemical properties prior to start of excavation.
- Recorded concentrations and pH values of groundwater chemical properties after backfill is completed and at six month intervals thereafter.
- One-year after backfill is completed:
    - If no negative trend is identified, inspection intervals can be increased to once per year.
    - If a negative trend is identified, need for dewatering provisions will be evaluated for other below-grade concrete structures and utilities.}

3.8.6 References

{**AASHTO, 2002.** Standard Specifications for Highway Bridges, 17th Edition, American Association of State and Highway Transportation Officials, September 2002.

ACI, 1997. Code Requirements for Nuclear Safety-Related Concrete Structures, ACI 349-97, American Concrete Institute, 1997.

ACI, 2001a. Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety-Related Concrete Structures, ACI 349-01/349-R01, American Concrete Institute, 2001.

ACI, 2001b. Guide to Durable Concrete, ACI 201.2R-01, American Concrete Institute, 2001.

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

ACI, 2007. Reinforced Concrete Design for Thermal Effects on Nuclear Power Plant Structures, ACI 349.1 R-07, American Concrete Institute, 2007.

AREVA, 2008. U. S. EPR Piping Analysis and Pipe Support Design, Revision 0, AREVA NP Inc., Topical Report ANP-10264NP-A, November 2008.

ASCE, 2000. Seismic Analysis of Safety-Related Nuclear Structures and Commentary, ASCE 4-98, American Society of Civil Engineers, 2000.

ASCE, 2005. Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-05, American Society of Civil Engineers, 2005.

ASME, 1994. Quality Assurance Requirements for Nuclear Facility Applications, ASME NQA-1-1994 Edition, American Society of Mechanical Engineers, 1994.

ASTM, 2005. Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for use in Concrete, ASTM C618-05, American Society for Testing and Materials, 2005.

ASTM, 2009. Standard Specification for Portland Cement, ASTM C150-09, American Society for Testing and Materials, 2009.

CFR, 2008. Requirements for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants, Title 10, Code of Federal Regulations, Part 50.65, 2008.

Dean, 1974. Evaluation and Development of Water Wave Theories for Engineering Application. Special Report No. I. Coastal Engineering Research Center, U.S. Army Corps of Engineers, November 1974.

IEEE, 2001. Standard Criteria for the Design, Installation, and Qualification of Raceway Systems for Class 1E Circuits for Nuclear Power Generating Stations, IEEE 628-2001, IEEE, 2001.

NRC, 1978a. Evaluations of Explosions Postulated To Occur on Transportation Routes Near Nuclear Power Plants, Regulatory Guide 1.91, Revision 1, U.S. Nuclear Regulatory Commission, February 1978.

NRC, 1978b. Inspection of Water-Control Structures Associated with Nuclear Power Plants, Regulatory Guide 1.127, Revision 1, U.S. Nuclear Regulatory Commission, March 1978.

NRC, 1997. Monitoring the Effectiveness of Maintenance at Nuclear Power Plants, Regulatory Guide 1.160, Revision 2, U.S. Nuclear Regulatory Commission, March 1997.

NRC, 2001. Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments), Regulatory Guide 1.142, Revision 2, U.S. Nuclear Regulatory Commission, November 2001.

NRC, 2007. NUREG-0800, Standard Review Plan, Section 3.8.4, "Other Seismic Category I Structures," Revision 2, U.S. Nuclear Regulatory Commission, March 2007.

USACE, 2006. Coastal Engineering Manual. Engineering Manual EM 1110-2-1100, U.S. Army Corps of Engineers, 2006.}

Table 3.8-1 {Static Frictional Parameters}

Interface	Friction Coefficient	Adhesion (ksf)
NI Common Basemat Structures		
Basemat — Mudmat	0.60	-
Mud mat — Sand	0.58	-
Sand — Waterproofing	0.52	-
Sand — Structural Fill	0.58	-
Structural Fill—Stratum IIb	0.47	1.0
EPGBs and ESWBs		
Basemat — Mudmat	0.60	-
Mudmat — Structural Fill	0.52	-
Structural Fill—Stratum IIb	0.47	1.0
CBIS		
Basemat — Mudmat	0.60	-
Mudmat — Stratum IIc	0.21	1.2
Side wall — Structural fill	0.58	-
UHS Electrical Building		
Basemat — Mudmat	0.60	-
Mudmat — Sand	0.58	-
Sand — Waterproofing	0.52	-
Sand — Structural fill	0.58	-
Structural Fill—Stratum IIc	0.21	1.2

Table 3.8-2 {Stability Evaluation Results for the CBIS and UHS Electrical Building}

Building	Load Combination (LC)	Factors of Safety (FOS)		
		Sliding	Overturning	Floatation
CBIS	D + H + W	103.0	75.2	-
	D + H + Wt	50.3	19.5	-
	D + H + E'	1.18	2.04	-
	D + F'	-	-	1.69
	D + H + SPH	6.17	19.4	-
	D + H + PMH	3.18	10.6	-
UHS EB	D + H + W	91.0	72.2	-
	D + H + Wt	32.1	22.7	-
	D + H + E'	1.10	2.06	-
	D + F'	-	-	1.30
	D + H + SPH	2.05	3.65	-
	D + H + PMH	1.10	2.16	-

Notes:

- For the CBIS, friction (traction) between side wall and backfill is utilized for LC (D+H+E').
- For the UHS EB, passive soil pressure is utilized for LC (D+H+E') and (D+H+PMH).
- For the CBIS, factor of safety against floatation (D+F') is governed by the PMH draw-down condition.

Table 3.8-3 {Bearing Capacity Evaluation Results for the CBIS and UHS Electrical Building}

Building	Load Combination	Bearing pressure (ksf)	
		Average	Maximum
CBIS	D + L + F	3.84	4.73
	D + L + F + W	3.83	4.74
	D + L + F + SPH	4.83	6.30
	D + L + F + Wt	3.84	4.79
	D + L + F + E'	4.47	5.89
	D + L + F + PMH	5.78	8.90
UHS EB	D + L + F	2.11	2.23
	D + L + F + W	2.11	2.25
	D + L + F + SPH	2.46	2.79
	D + L + F + Wt	2.11	2.29
	D + L + F + E'	2.53	2.82
	D + L + F + PMH	3.21	3.84

Note:

Static and dynamic bearing capacities are 12 ksf and 18 ksf, respectively, for the CBIS and 11 ksf and 16.5 ksf, respectively, for the UHS EB (See Table 2.5-65).

Table 3.8-4 {Factors of Safety for NI Common Basemat Structure, EPGB, and ESWB under SSE Loading}

Building	Sliding	Overturning
NI Common Basemat Structure	1.88	4.50
EPGB	1.77	3.17
ESWB	3.19	4.0

Table 3.8-5 {Observed Chemical Properties of Groundwater}

Properties	Surficial aquifer	Upper Chesapeake unit
pH (average)	5.2	7.4
Sulfate (ppm, maximum)	68.6	365
Chloride (ppm, maximum)	47.4	370

Notes:

Sulfate and chloride concentrations indicate the maximum observed values at the powerblock and intake areas.

ppm = parts per million.

Figure 3.8-1 {Schematic Site Plan of Seismic Category I Buried Utilities (Electrical Duct Banks)}

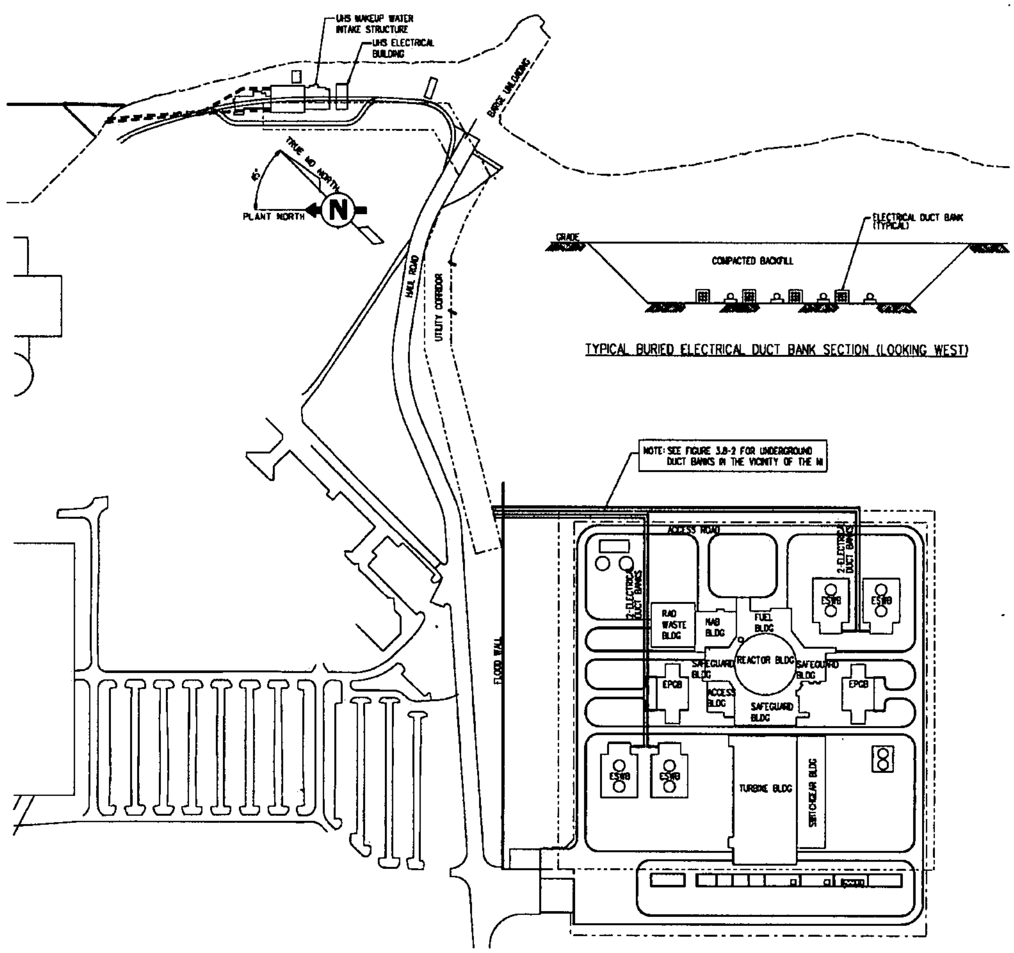


Figure 3.8-2 {Schematic Site Plan of Seismic Category I Buried Utilities at the NI (Electrical Duct Banks)}

This figure contains security related information and has been withheld under

10 CFR 2.390 (d)(1)

See Part 9 of the COLA Application

NOTE:

This figure is unchanged

Figure 3.8-3 {Schematic Site Plan of Seismic Category I Buried Utilities (Underground Piping)}

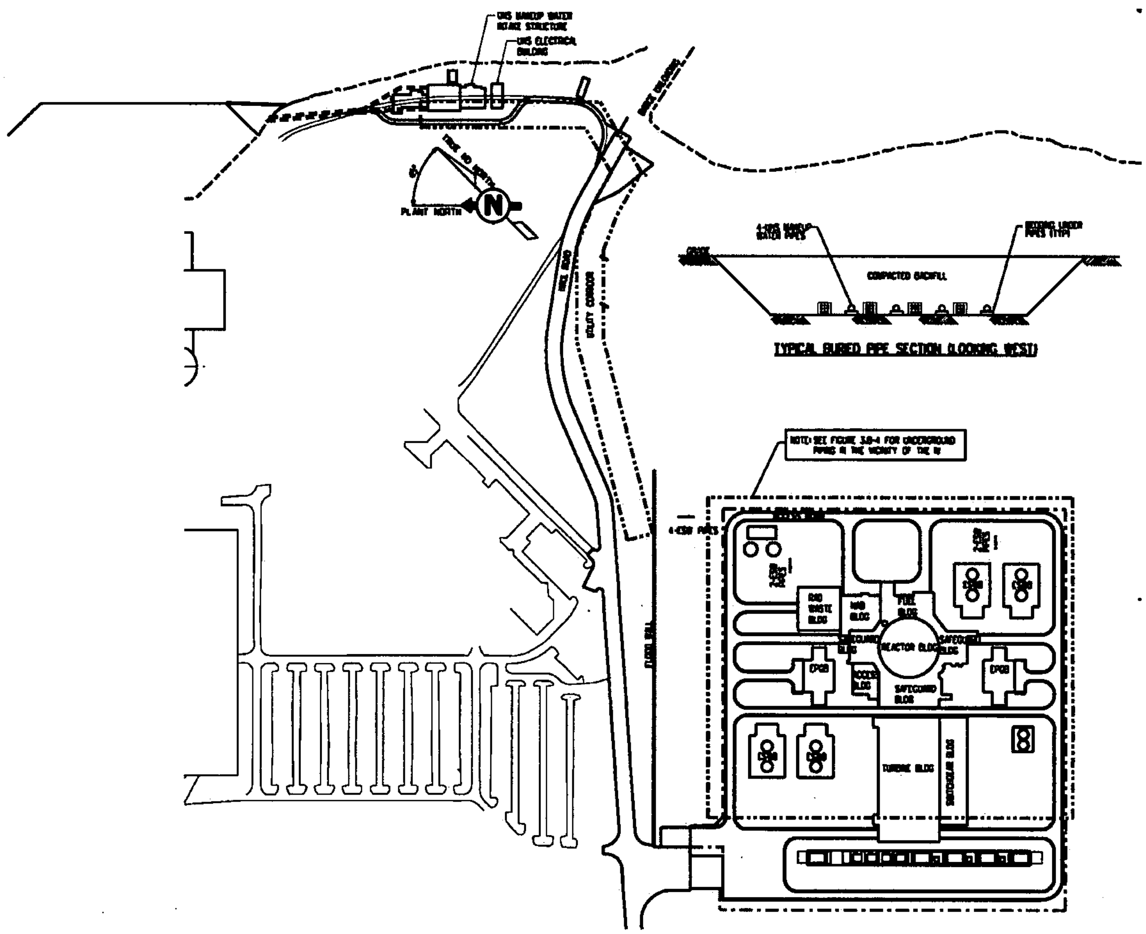


Figure 3.8-4 {Schematic Site Plan of Seismic Category I Buried Utilities (Underground Piping)}

This figure contains security related information and has been withheld under

10 CFR 2.390 (d)(1)

See Part 9 of the COLA Application

NOTE:

This figure is unchanged

Figure 3.8-5 {Isometric View of the Basemat Finite Element Mesh (half model) for the CWS Makeup Water Intake Structure, Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building}

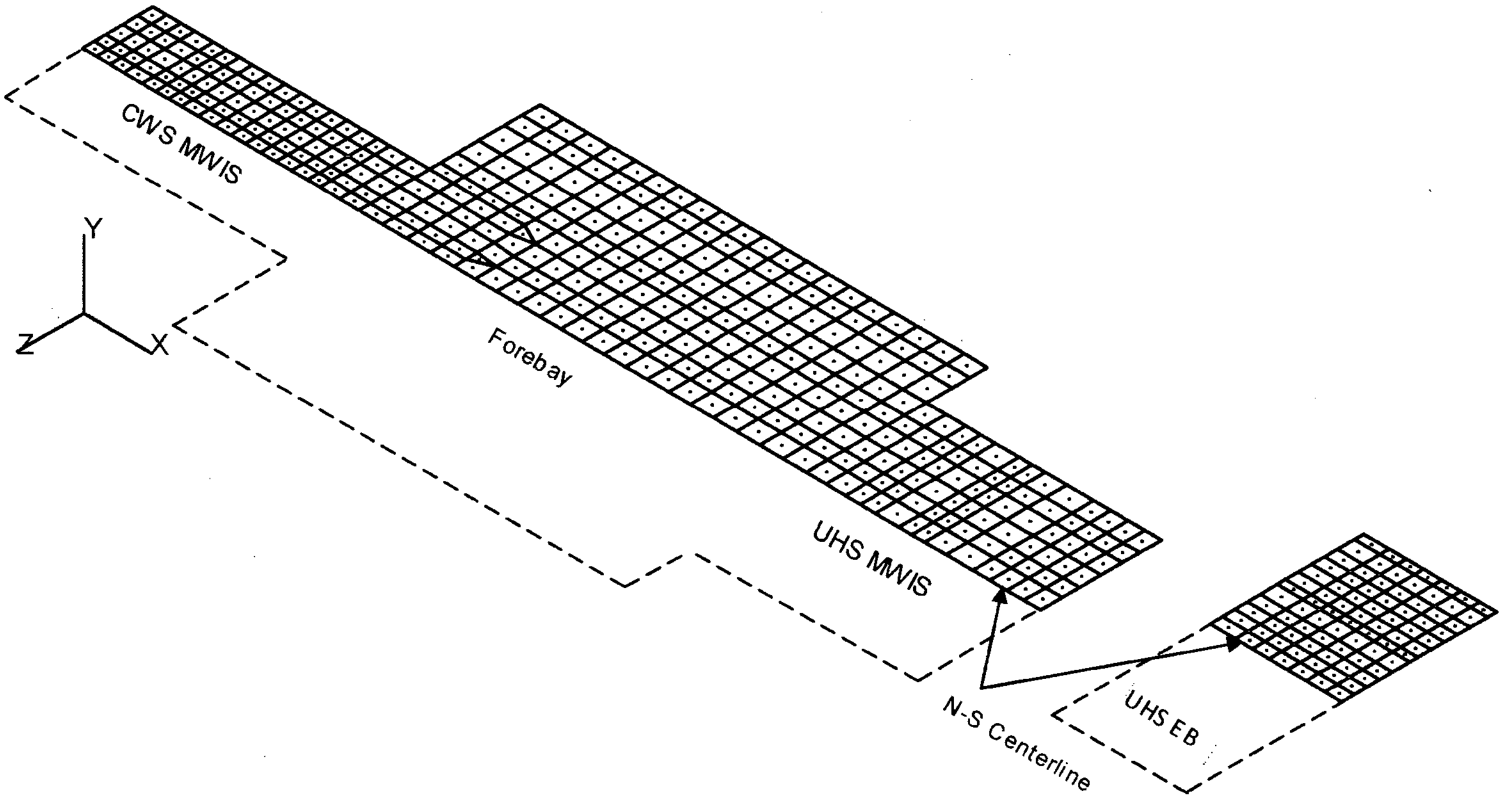
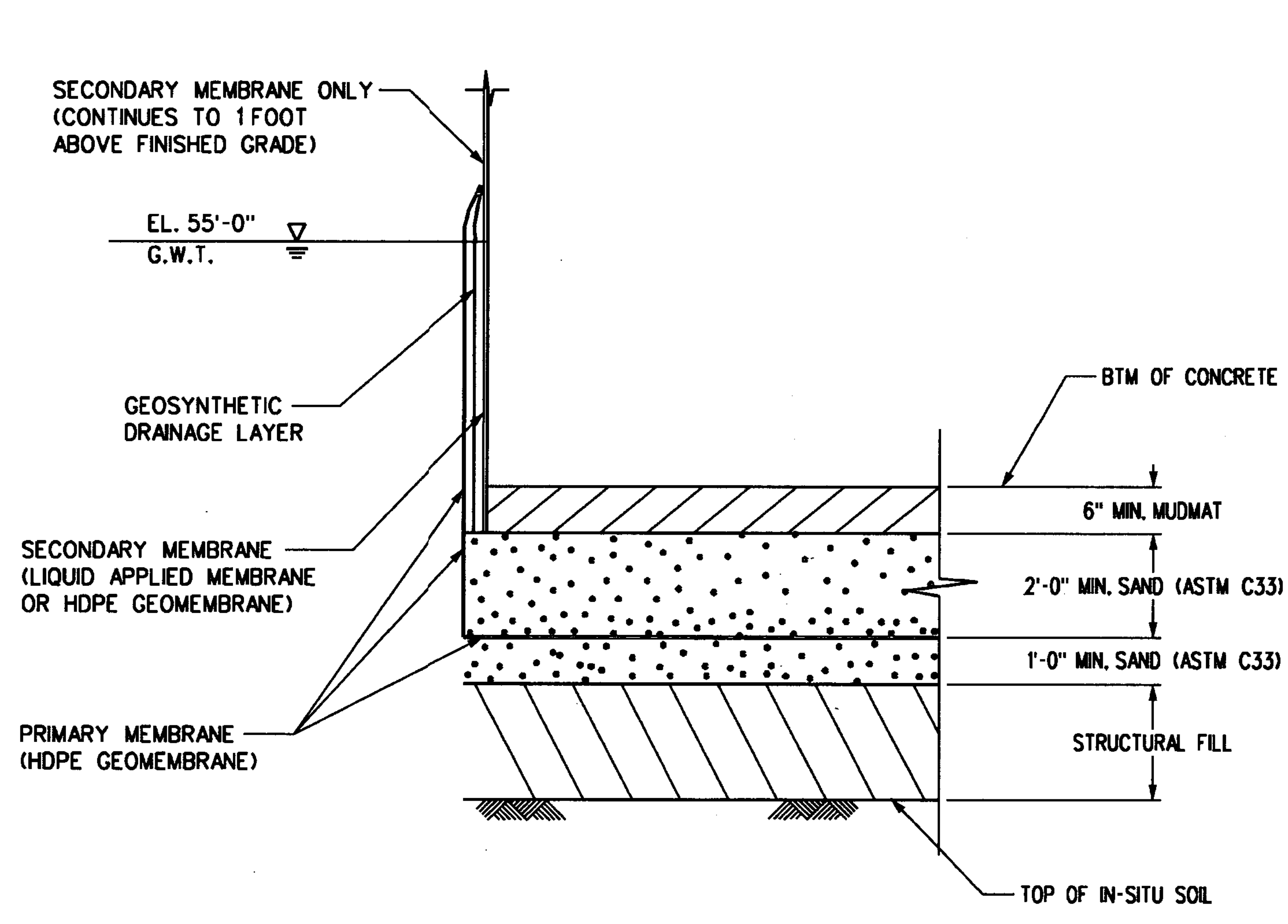


Figure 3.8-6 {Conceptual Configuration of Waterproofing Membrane}



3E CRITICAL SECTIONS FOR SAFETY-RELATED CATEGORY I STRUCTURES

This section of the U.S. EPR FSAR is incorporated by reference, with the following supplements.

The U.S. EPR FSAR contains the following COL item in Appendix 3E:

> A COL applicant that references the U.S. EPR design certification will address critical sections relevant to site-specific Seismic Category I structures.

This COL item is addressed as follows:

{Section 3E.4 of Appendix 3E provides the discussion regarding the critical sections of the site-specific Seismic Category I Structures:

- Forebay
- Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS)
- UHS Electrical Building (EB)}

3E.1 NUCLEAR ISLAND STRUCTURES

No departures or supplements.

3E.2 EMERGENCY POWER GENERATING BUILDINGS

No departures or supplements.

3E.3 ESSENTIAL SERVICE WATER BUILDINGS

No departures or supplements.

3E.4 {FOREBAY, UHS MAKEUP WATER INTAKE STRUCTURE AND UHS ELECTRICAL BUILDING

This section is a supplement to U.S. EPR FSAR Appendix 3E.

3E.4.1 Structural Description and Geometry

The General Arrangement plans and elevations of the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are provided as Figures 9.2-4, 9.2-5 and 9.2-6. A general description of the structures is provided in Section 3.8.4.1.11. Section 3.8.5.1.4 provides additional details regarding the basemats.

A Foundation Plan for the Forebay and UHS Makeup Water Intake Structure at Elevation -22 ft 6 in (-6.9 m) is provided as Figure 3E.4-1. For the UHS Electrical Building, the Foundation Plan at Elevation -5 ft 6 in (-1.7 m) is provided as Figure 3E.4-2. As described in Section 3E.4.4, the following critical structural elements are selected for design based on their location, dimension, support conditions, and applied loads:

- Basemat of the Forebay (Figure 3E.4-1).

- Basemat of the UHS Makeup Water Intake Structure (Figure 3E.4-1).
- Basemat of the UHS Electrical Building (Figures 3E.4-2).
- Long wall of the Forebay (Figures 3E.4-1 and 3E.4-5).
- Side wall of the UHS Makeup Water Intake Structure water basin (Figures 3E.4-1 and 3E.4-4).
- Side wall of the UHS Makeup Water Intake Structure pump house (Figure 3E.4-4).
- North wall of the UHS Electrical Building (Figures 3E.4-2 and 3E.4-7).

Forebay and UHS Makeup Water Intake Structure share a common basemat, as described in Section 3.8.5.1.4. Additional descriptions of the critical structural elements are provided in Section 3E.4.4.

3E.4.2 Material Properties

Concrete and reinforcing steel materials for the site-specific Seismic Category I structures conform to the requirements of U.S. EPR FSAR Sections 3.8.4.6 and 3.8.5.6. The following material properties are used in critical section design:

- Concrete
  - Compressive strength (fc'): 5000 psi (34.5 MPa) minimum at 28 days
  - Modulus of elasticity (E): 4287 ksi (2.96 x 104 MPa)
  - Shear modulus (G): 1832 ksi (1.26 x 104 Mpa)
  - Poisson's ratio: 0.17
- Reinforcement
  - Yield stress (fy): 60 ksi (413.7 MPa)

General description of foundation soil is provided in Section 2.5.4. Soil properties and ground water table for calculating lateral earth pressure are described in Section 3.8.4.3.1.

3E.4.3 Structural Loads and Load Combinations

Structural loads and load combinations for the design of site-specific Seismic Category I structures are specified in Sections 3.8.4.3.1 and 3.8.4.3.2, respectively. For convenience, the basic load combinations used for concrete design are repeated below:

- Normal: 1.4(D + F) + 1.7(L + H)
- Wind: 1.4(D + F) + 1.7(L + H + W)
- SPH: 1.4(D + F) + 1.7(L + H + SPH)
- SSE: D + L + H + F + E'
- Tornado: D + L + H + F + Wt
- PMH: D + L + H + F + PMH

Where,

D	=	Dead load
L	=	Live load
F	=	Hydrostatic load from water inside structures
H	=	Lateral earth pressure including load due to water outside structures and compaction pressures.
W	=	Normal wind load
Wt	=	Tornado wind load
SPH	=	Standard Project Hurricane load
PMH	=	Probable Maximum Hurricane load
E'	=	Safe Shutdown Earthquake (SSE) load

Additional load combinations for stability and bearing pressure evaluation are specified in Section 3.8.5.3.

3E.4.4 Structural Analysis and Design

The analysis and design procedures for Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are presented in Sections 3.8.4.4.7 and 3.8.5.4.6, including the procedures for stability and bearing pressure evaluation. Structural acceptance criteria are presented in Sections 3.8.4.5 and 3.8.5.5. Selection and design of critical elements is further discussed in this section.

Selection of Critical Elements

The following critical sections are selected for design. Clear dimensions are used in the descriptions.

- *Foundation Basemats (Figures 3E.4-1 and 3E.4-2)*: Foundation basemats transfer all applicable vertical and horizontal structural loads to the supporting soil. Therefore, basemats of the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are selected as critical structural elements. Basemats of Forebay and UHS Makeup Water Intake Structure are part of the common basemat of the CBIS and are integrally connected, while the UHS Electrical Building sits on a separate basemat at a higher elevation. Further descriptions of the basemats are provided in Section 3.8.5.1.4.

- *Forebay Long Wall (Figures 3E.4-1 and 3E.4-5)*: Long walls in the plant north-south direction are subject to large lateral earth pressure. Each wall is 100 ft (30.5 m) long, 34 ft (10.4 m) high, and 4.5 ft (1.4 m) thick. Due to its length and support conditions, the center portion of each wall behaves like a cantilever making it a critical element.

- *UHS Makeup Water Intake Structure Side Walls (Figures 3E.4-1 and 3E.4-4)*: Below-grade side walls of the UHS Makeup Water Intake Structure in the plant north-south direction are subject to large lateral earth pressures. Each wall is 50.5 ft (15.4 m) long, 31 ft (9.4 m) high, and 4 ft (1.2 m) thick. The loading condition is more critical when the adjacent pump bay is emptied for maintenance. Side walls of the Pump House, which sit above the operating deck

(Elevation 11'-6" (3.51 m)), are 53.5 ft (16.3 m) long, 13 ft (4.0 m) tall, and 2 ft (0.61 m) thick and subject to large hurricane wave pressures.

- *UHS Electrical Building North Wall (Figures 3E.4-2 and 3E.4-7)*: The exterior wall on the north side of the building is not supported by any interior wall and is considered critical under lateral earth pressure. The wall is about 70 ft (21.3 m) long, 14 ft (4.3 m) high, and 2 ft (0.6 m) thick.

Design of Critical Elements

Structural analysis and design of the aforementioned critical sections are performed using the procedures outlined in Section 3.8.4.4.7. Each critical concrete section is designed for combined axial force and bending moment, shear friction, in-plane and outof- plane shear according to the applicable provisions of ACI 349-01 (ACI, 2001).

As stated in Section 3.8.4.4.7, design forces and moments are calculated using the finite element results from SASSI for seismic loads and GT STRUDL for non-seismic loads. Design for combined axial force (P) and bending moment (M) is based on P-M interaction of element level results. Design for shear friction, in-plane, and out-of-plane shear is based on section cuts at critical locations of a wall or basemat.

The following provisions from ACI 349-01 (ACI, 2001) are used for design:

- *Axial force and bending moment*: Sections 7.12, 9.3, 10.2, 10.3, 14.3, and 21.6.

- *Shear friction*: Section 11.7. A friction coefficient of 1.0 is used for concrete placed against hardened concrete with surface intentionally roughened. The beneficial effect of compression is ignored.

- *In-plane shear*: Section 11.10 (non-seismic loads) and Section 21.6 (seismic loads). As discussed in U.S. EPR FSAR Section 3.8.4.4.1, a shear strength reduction factor of 0.85 is used.

- *Out-of-plane shear*: Sections 11.1, 11.3, 11.5, and 11.12.

For all section cuts, the design reinforcement is based on the sum of reinforcement required for in-plane shear and combined axial force and bending moment. For a section cut subject to shear friction requirement, the reinforcement provided for combined axial force and bending moment is checked for shear friction and increased, if necessary. Minimum reinforcement required by the code is satisfied. Maximum concrete section strengths limited by the code are also satisfied.

Results of critical section design in terms of the demand to capacity ratios and estimated reinforcements are presented in Section 3E.4.5.

3E.4.5 Summary of Results

Arrangement of main reinforcement for the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building is shown in Figures 3E.4-3 through 3E.4-7. Note that supplementary shrinkage and temperature reinforcement is not shown for clarity, but it will be provided where required. The maximum demand to capacity ratios are presented in Tables 3E.4-1 through 3E.4-4 for various design load combinations.

Stability evaluation results of the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building are presented in Table 3.8-2. Bearing pressure calculation results are presented in Table 3.8-3. These results are discussed in Section 3.8.5.5.4.

3E.4.6 Conclusions

The critical sections of the Forebay, UHS Makeup Water Intake Structure, and UHS Electrical Building have adequate strength to resist the structural loads from various design basis events. The structures satisfy the minimum required factors of safety against sliding, overturning and floatation loading conditions. The foundation soil has adequate capacity to resist bearing pressure.

3E.4.7 References

ACI, 2001. Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary on Code Requirements for Nuclear Safety-Related Concrete Structures, ACI 349-01/349R-01, American Concrete Institute, 2001.

Table 3E.4-1 – {Demand and Capacity for In-Plane Shear}

Section	Load Combination [(a)]	Vu [(b)] (kip)	φVc [(c)] (kip)	D/C [(d)]
Forebay Long Wall	Normal	1567	12333	0.13
	Wind	1602	12330	0.13
	SPH	2269	12272	0.18
	SSE	3434	11684	0.29
	Tornado	1035	11961	0.09
	PMH	2092	11992	0.17
UHS MWIS Water Basin Side Wall	Normal	1725	5597	0.31
	Wind	1728	5596	0.31
	SPH	1870	5590	0.33
	SSE	1892	5141	0.37
	Tornado	1014	5592	0.18
	PMH	1379	5654	0.24
UHS MWIS Pump House Side Wall	Normal	43	1729	0.03
	Wind	56	1725	0.03
	SPH	277	1793	0.15
	SSE	447	1584	0.28
	Tornado	27	1704	0.02
	PMH	324	1884	0.17
UHS EB North Wall	Normal	70	1114	0.06
	Wind	73	1113	0.07
	SPH	151	1098	0.14
	SSE	191	1039	0.18
	Tornado	52	1107	0.05
	PMH	164	1109	0.15

Notes:

(a) Load combinations are defined in Section 3E.4.3
(b) Vu = Maximum in-plane shear demand
(c) φVc = Nominal in-plane shear strength due to concrete as defined in Section 3E.4.4
(d) D/C = Demand/Capacity, i.e. Vu/φVn

Table 3E.4-2 – {Demand and Capacity for Out-of-Plane Shear}

Section	Load [(a)] Combination	Vu [(b)] (kip)	φVc [(c)] (kip)	D/C [(d)]
Common Basemat	Normal	6454	8741	0.74
	Wind	6465	8741	0.74
	SPH	6182	8741	0.71
	SSE	5004	8741	0.57
	Tornado	4238	8741	0.48
	PMH	4147	7137	0.58
UHS EB Basemat	Normal	654	5934	0.11
	Wind	661	5934	0.11
	SPH	821	5934	0.14
	SSE	725	5934	0.12
	Tornado	412	5934	0.07
	PMH	714	5934	0.12
Forebay Long Wall	Normal	6439	7797	0.83
	Wind	6446	7797	0.83
	SPH	5878	7797	0.75
	SSE	5271	7797	0.68
	Tornado	4679	7797	0.60
	PMH	3229	7797	0.41
UHS MWIS Water Basin Side Wall	Normal	2461	3710	0.66
	Wind	2464	3710	0.66
	SPH	1705	3710	0.46
	SSE	1622	3710	0.44
	Tornado	1336	3710	0.36
	PMH	1152	3710	0.31
UHS MWIS Pump House Side Wall	Normal	107	1031	0.10
	Wind	128	1031	0.12
	SPH	648	1031	0.63
	SSE	83	1031	0.08
	Tornado	79	1031	0.08
	PMH	842	1031	0.82
UHS EB North Wall	Normal	1490	2091	0.71
	Wind	1499	2091	0.72
	SPH	1932	2091	0.92
	SSE	1815	2091	0.87
	Tornado	847	2091	0.41
	PMH	1392	2091	0.67

Notes:

(a) Load combinations are defined in Section 3E.4.3
(b) Vu = Maximum out-of-plane shear demand
(c) φVc = Nominal out-of-plane shear strength due to concrete as defined in Section 3E.4.4
(d) D/C = Demand/Capacity, i.e. Vu/φVc

Table 3E.4-3 – {Demand and Capacity for Combined Moment and Axial Force}
Page 1 of 4

(a) CBIS Common Basemat (5 ft thick)
(for areas where 1 layer of #11 @ 7" each face is required)

Section Direction	Load [a] Combination	Mu [b] (kip-ft)	Pu [c] (kip)	φMn [d] (kip-ft)	φPn [e] (kip)	D/C [f]
N-S	Normal	444	0	544	1881	0.82
	Wind	446	0	544	1881	0.82
	SPH	473	0	544	1881	0.87
	SSE	263	15	573	1881	0.46
	Tornado	238	0	544	1881	0.44
	PMH	190	0	544	1881	0.35
E-W	Normal	259	0	544	1881	0.48
	Wind	262	0	544	1881	0.48
	SPH	389	0	544	1881	0.71
	SSE	83	-65	421	-245	0.27
	Tornado	157	0	544	1881	0.29
	PMH	195	0	544	1881	0.36

(b) CBIS Common Basemat (5 ft thick)
(for areas where 2 layers of #11 @ 7" each face is required)

Section Direction	Load [a] Combination	Mu [b] (kip-ft)	Pu [c] (kip)	φMn [d] (kip-ft)	φPn [e] (kip)	D/C [f]
N-S	Normal	1043	0	1083	2000	0.96
	Wind	1045	0	1083	1999	0.97
	SPH	845	0	1083	2048	0.78
	SSE	689	18	1117	2048	0.62
	Tornado	629	0	1083	2048	0.58
	PMH	458	0	1083	2048	0.42
E-W	Normal	404	0	1083	2048	0.37
	Wind	404	0	1083	2048	0.37
	SPH	656	0	1083	2048	0.61
	SSE	261	20	1121	2048	0.23
	Tornado	256	0	1083	2048	0.24
	PMH	480	0	1083	2048	0.44

Table 3E.4-3 – {Demand and Capacity for Combined Moment and Axial Force}

(c) UHS EB Common Basemat (5 ft thick)
(1 layer #9 @ 12" each face)

Section Direction	Load [(a)] Combination	Mu [(b)] (kip-ft)	Pu [(c)] (kip)	φMn [(d)] (kip-ft)	φPn [(e)] (kip)	D/C [(f)]
N-S	Normal	52	0	199	1776	0.26
	Wind	52	0	199	1776	0.26
	SPH	69	0	199	1776	0.35
	SSE	63	3	204	1776	0.31
	Tornado	28	0	199	1776	0.14
	PMH	45	0	199	1776	0.23
E-W	Normal	74	0	199	1776	0.37
	Wind	75	0	199	1776	0.37
	SPH	104	0	199	1776	0.52
	SSE	96	10	217	1776	0.44
	Tornado	41	0	199	1776	0.21
	PMH	78	0	199	1776	0.39

(d) Forebay Long Wall (4.5 ft thick)
(for areas where 1 layer of #11 @ 7" each face is required)

Section Direction	Load [(a)] Combination	Mu [(b)] (kip-ft)	Pu [(c)] (kip)	φMn [(d)] (kip-ft)	φPn [(e)] (kip)	D/C [(f)]
Vertical	Normal	487	64	597	1709	0.81
	Wind	488	64	598	1709	0.82
	SPH	361	46	566	1709	0.64
	SSE	311	37	551	1709	0.56
	Tornado	296	42	559	1709	0.53
	PMH	278	37	551	1709	0.50
Horizontal	Normal	290	7	501	1709	0.58
	Wind	290	7	501	1709	0.58
	SPH	245	29	539	1709	0.45
	SSE	206	7	500	1709	0.41
	Tornado	176	8	503	1709	0.35
	PMH	147	40	557	1709	0.26

Table 3E.4-3 – {Demand and Capacity for Combined Moment and Axial Force}

Page 3 of 4

(e) Forebay Long Wall (4.5 ft thick)
(for areas where 2 layers of #11 @ 7" each face are required)

Section Direction	Load [(a)] Combination	Mu [(b)] (kip-ft)	Pu [(c)] (kip)	φMn [(d)] (kip-ft)	φPn [(e)] (kip)	D/C [(f)]
Vertical	Normal	960	125	1174	1764	0.82
	Wind	961	125	1173	1763	0.82
	SPH	805	161	1234	1858	0.65
	SSE	627	103	1138	1876	0.55
	Tornado	599	93	1121	1876	0.53
	PMH	551	113	1155	1876	0.48
Horizontal	Normal	915	66	1075	1792	0.85
	Wind	915	66	1075	1791	0.85
	SPH	798	89	1114	1862	0.72
	SSE	629	61	1067	1876	0.59
	Tornado	574	55	1058	1876	0.54
	PMH	448	61	1068	1876	0.42

(f) UHS MWIS Water Basin Side Wall (4 ft thick)
(1 layer of #11 @ 10" each face)

Section Direction	Load [(a)] Combination	Mu [(b)] (kip-ft)	Pu [(c)] (kip)	φMn [(d)] (kip-ft)	φPn [(e)] (kip)	D/C [(f)]
Vertical	Normal	425	140	510	1488	0.83
	Wind	430	142	513	1488	0.84
	SPH	520	170	555	1488	0.94
	SSE	324	62	394	1488	0.82
	Tornado	285	94	442	1488	0.65
	PMH	558	182	573	1475	0.97
Horizontal	Normal	190	34	353	1488	0.54
	Wind	191	34	353	1488	0.54
	SPH	34	-101	151	-179	0.56
	SSE	144	12	319	1488	0.45
	Tornado	116	17	326	1488	0.36
	PMH	177	35	353	1488	0.50

Table 3E.4-3 – {Demand and Capacity for Combined Moment and Axial Force}

(g) UHS MWIS Pump House Side Wall (2 ft thick)
(1 layer #9 @ 10" each face)

Section Direction	Load [(a)] Combination	Mu [(b)] (kip-ft)	Pu [(c)] (kip)	φMn [(d)] (kip-ft)	φPn [(e)] (kip)	D/C [(f)]
Vertical	Normal	2	-13	85	-127	0.10
	Wind	2	-15	84	-127	0.12
	SPH	3	-27	75	-125	0.21
	SSE	1	-7	89	-128	0.06
	Tornado	4	14	105	760	0.04
	PMH	15	14	105	760	0.14
Horizontal	Normal	1	-55	55	-128	0.43
	Wind	2	-58	53	-127	0.45
	SPH	20	-81	36	-102	0.79
	SSE	2	-38	67	-127	0.30
	Tornado	13	3	97	760	0.13
	PMH	32	-27	75	-85	0.43

(h) UHS EB North Wall (2 ft thick)
(1 layer #9 @ 12" each face)

Section Direction	Load [(a)] Combination	Mu [(b)] (kip-ft)	Pu [(c)] (kip)	φMn [(d)] (kip-ft)	φPn [(e)] (kip)	D/C [(f)]
Vertical	Normal	29	15	89	748	0.33
	Wind	30	15	89	748	0.34
	SPH	55	27	98	748	0.56
	SSE	32	14	89	748	0.37
	Tornado	17	11	86	748	0.20
	PMH	50	30	100	748	0.50
Horizontal	Normal	58	12	87	748	0.67
	Wind	59	12	87	748	0.68
	SPH	87	14	88	748	0.99
	SSE	70	9	85	748	0.82
	Tornado	33	8	84	748	0.39
	PMH	69	13	88	748	0.78

Notes:

(a) Load combinations are defined in Section 3E.4.3
(b) Mu = Bending moment demand
(c) Pu = Axial force demand (positive for compression)
(d) φMn = Bending moment capacity
(e) φPn = Axial force capacity
(f) D/C = Demand/capacity, larger of Mu/φMn and Pu/φPn

Table 3E.4-4 – {Demand and Capacity for Shear Friction}

Section	Load [(a)] Combination	Nu [(b)] (kip)	Vu [(c)] (kip)	φVn [(d)] (kip)	D/C [(e)]
Forebay Long Wall (2 layers of #11@7")	Normal	-6620	7852	29733	0.26
	Wind	-6605	7859	29733	0.26
	SPH	-6365	6247	29733	0.21
	SSE	-3911	5354	29733	0.18
	Tornado	-4432	4782	29733	0.16
	PMH	-4778	3794	29733	0.13
UHS MWIS Water Basin Side Wall (2 layers of #11@10")	Normal	-1936	2982	11266	0.26
	Wind	-1934	2982	11266	0.26
	SPH	-1662	2525	11266	0.22
	SSE	-1701	2063	11266	0.18
	Tornado	-1175	1671	11266	0.15
	PMH	-1538	1771	11266	0.16
UHS MWIS Pump House Side Wall (2 layers of #9@10")	Normal	-306	111	4468	0.02
	Wind	-280	131	4468	0.03
	SPH	-638	651	4468	0.15
	SSE	91	423	4390	0.10
	Tornado	-176	79	4468	0.02
	PMH	-1124	842	4468	0.19
UHS EB North Wall (2 layers of #9@12")	Normal	-790	1501	7548	0.20
	Wind	-777	1511	7548	0.20
	SPH	-748	1972	7548	0.26
	SSE	-978	1821	7548	0.24
	Tornado	-583	848	7548	0.11
	PMH	-792	1399	7548	0.19

Notes:

(a) Load combinations are defined in Section 3E.4.3
(b) Nu = Normal force on friction interface (positive for tension)
(c) Vu = Shear demand, vector sum of in-plane and out-of-plane shear
(d) φVn = Nominal shear friction strength
(e) D/C = Demand/Capacity, i.e. Vu/φVn

Figure 3E.4-1 {Foundation Plan for the Forebay and UHS Makeup Water Intake Structure @ Elevation -22.5 ft (-6.86 m)}

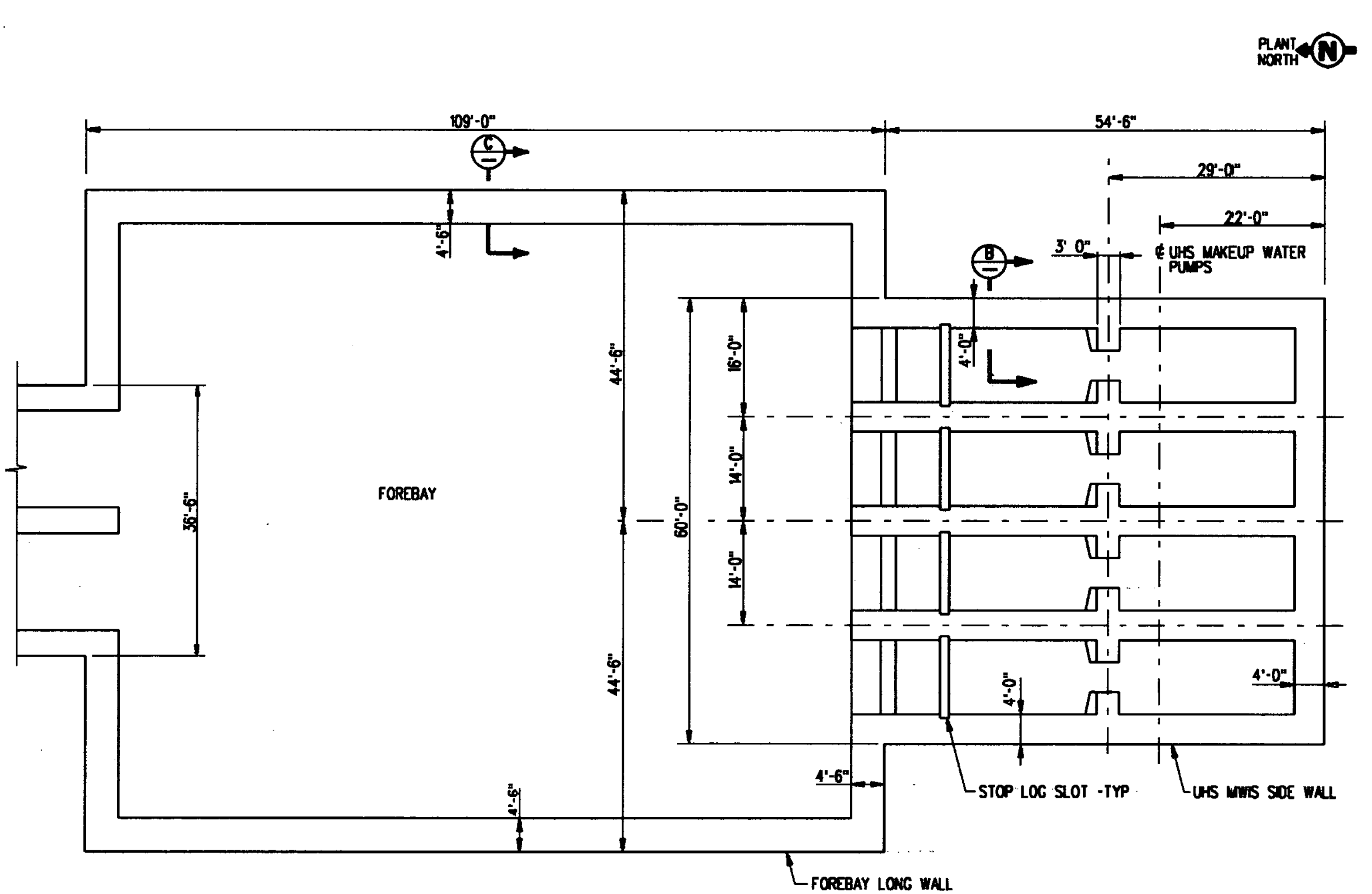


Figure 3E.4-2 {Foundation Plan for the UHS Electrical Building @ Elevation -5.5 ft (-1.68 m)}

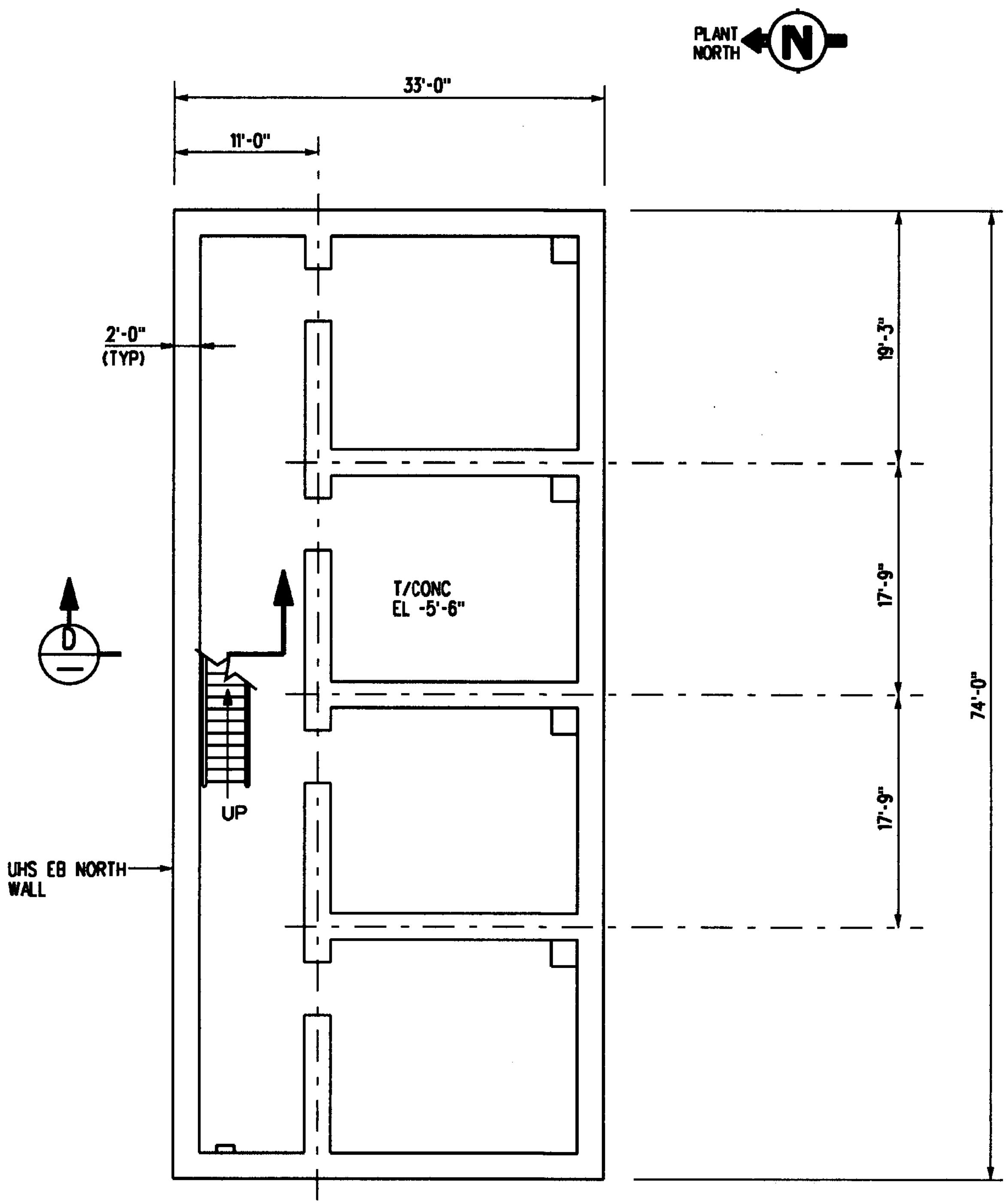
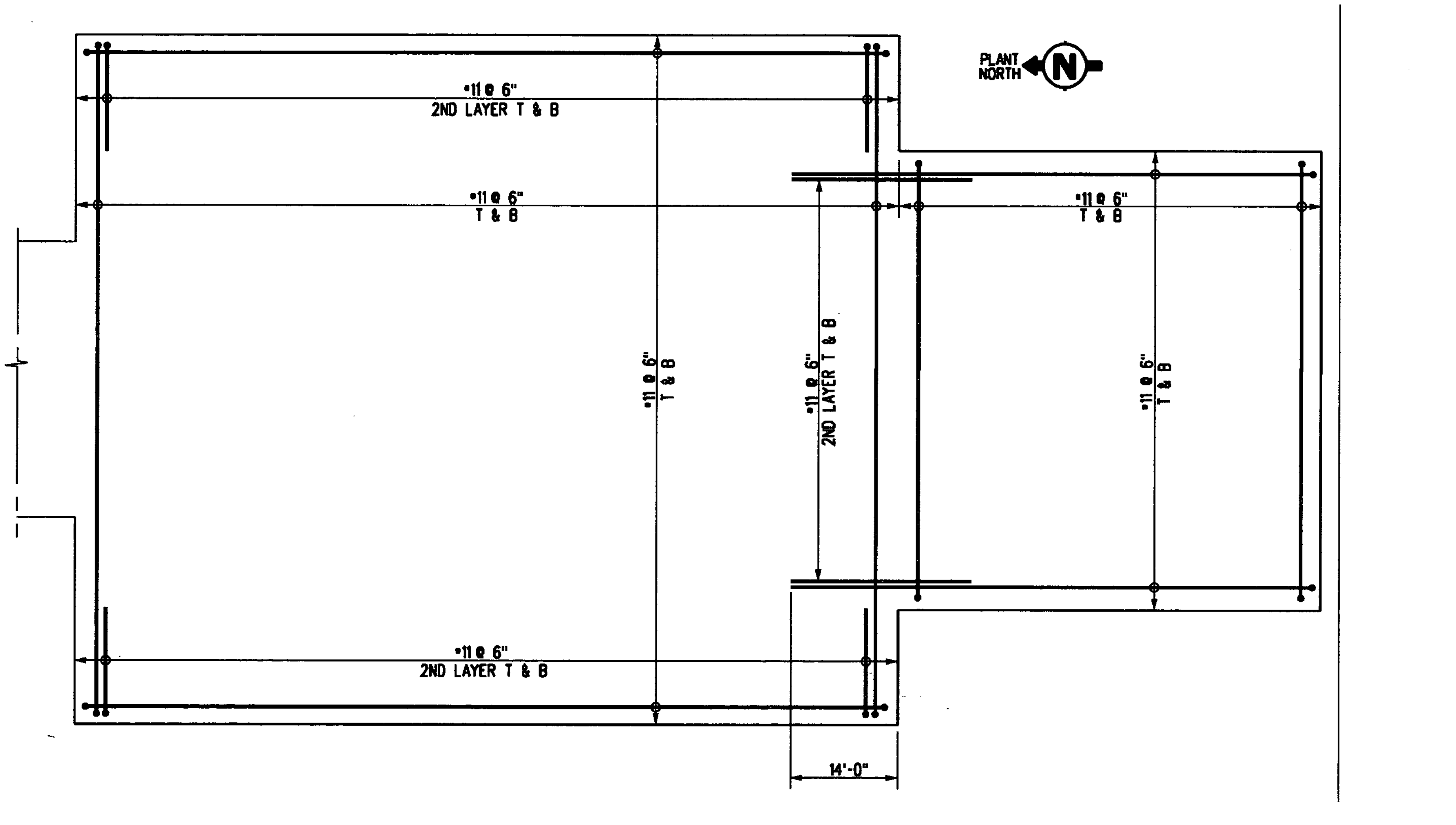
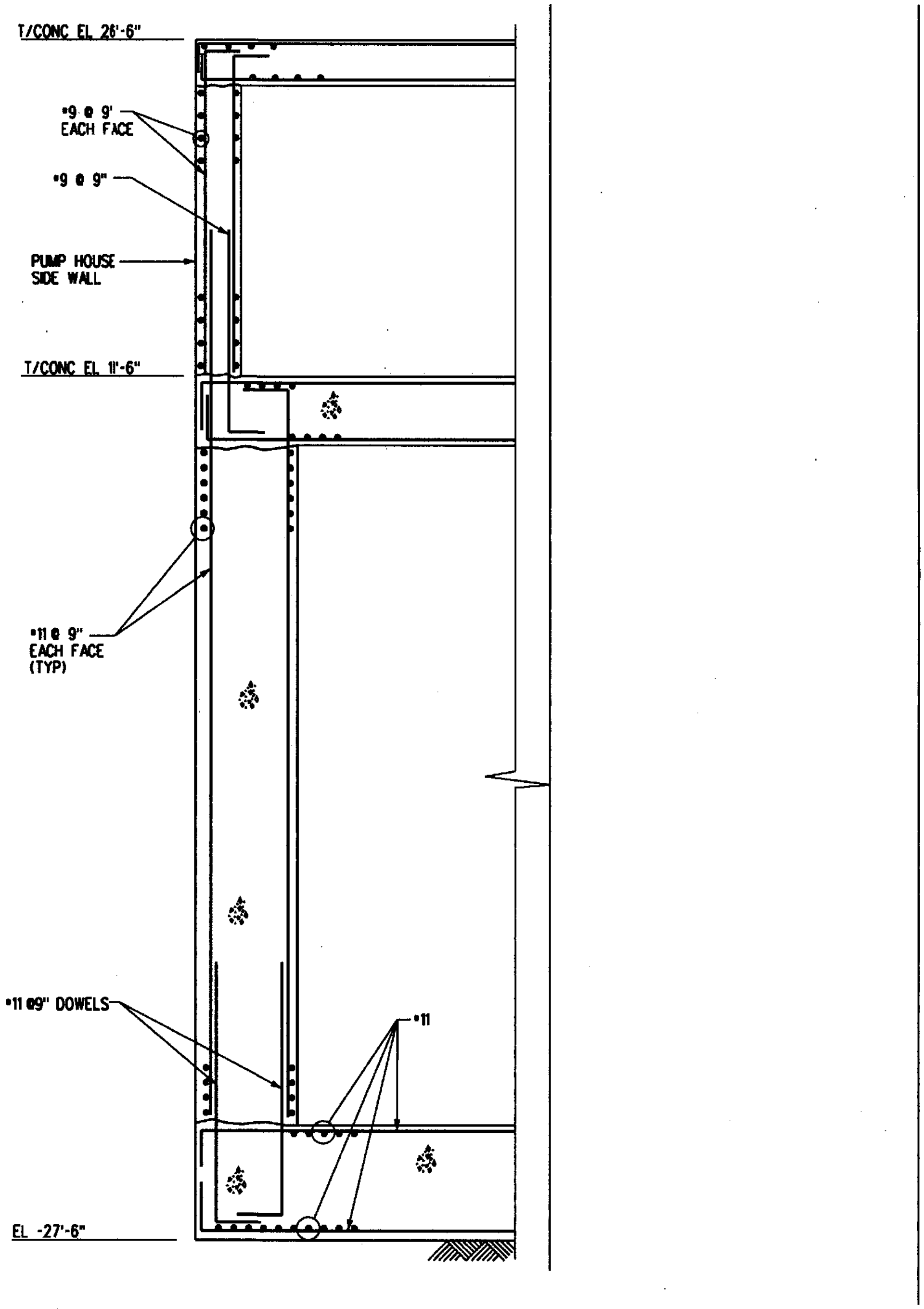


Figure 3E.4-3 {Reinforcement for Forebay and UHS Makeup Water Intake Structure Basemat}



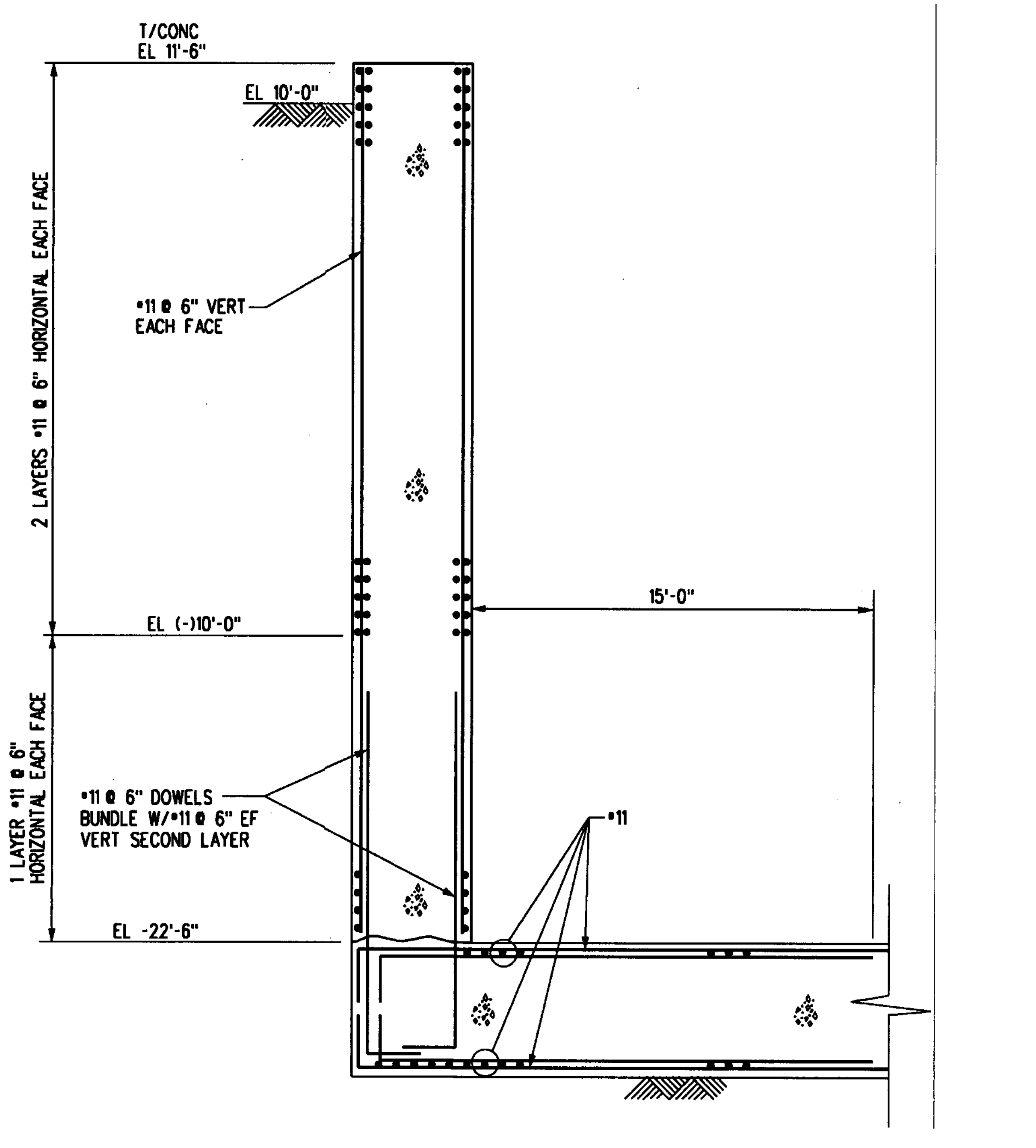
Note: Required rebar spacing is 7" (Table 3E.4-3 (a), (b)).

Figure 3E.4-4 {Reinforcement for Forebay and UHS Makeup Water Intake Structure Walls - UHS Makeup Water Intake Structure Side Wall (Section B)}



Note: Required rebar spacing for #11 is 10" (Table 3E.4-3 (f), (g)).

Figure 3E.4-5 {Reinforcement for Forebay and UHS Makeup Water Intake Structure Walls - Forebay Long Wall (Section C)}



Note: Required rebar spacing is 7" (Table 3E.4-3 (d), (e)).

Figure 3E.4-6 {Reinforcement for UHS Electrical Building Basemat}

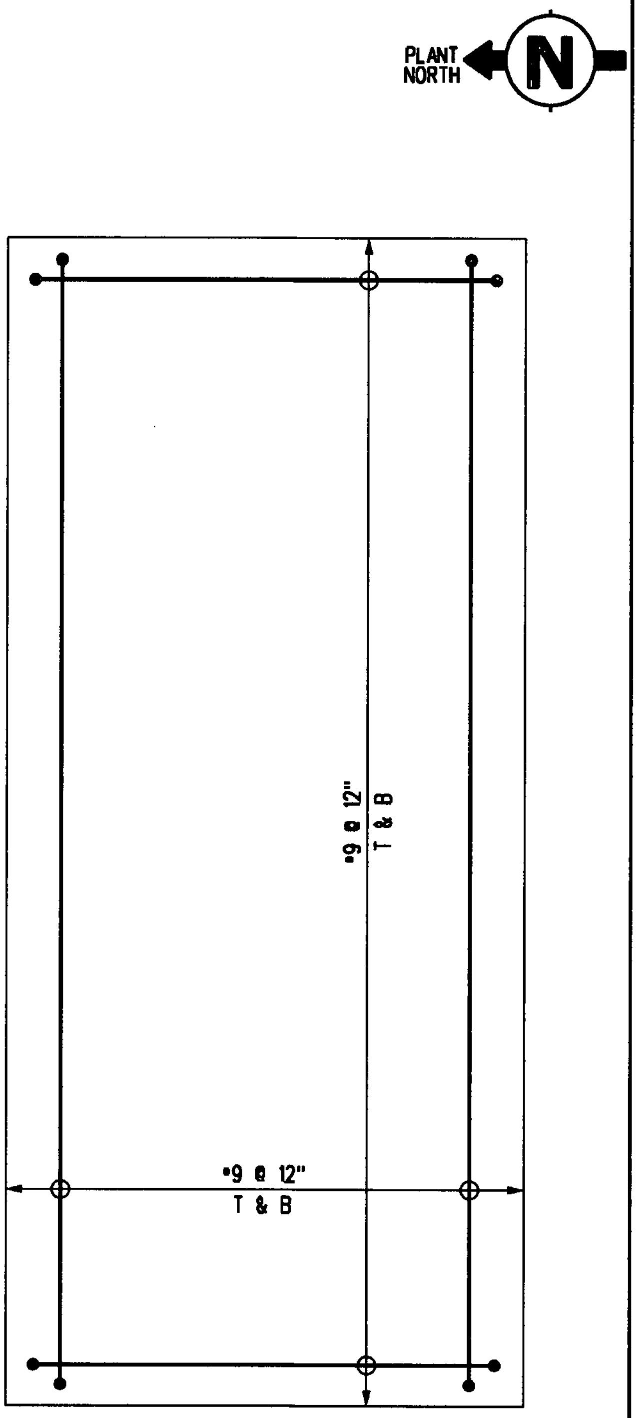
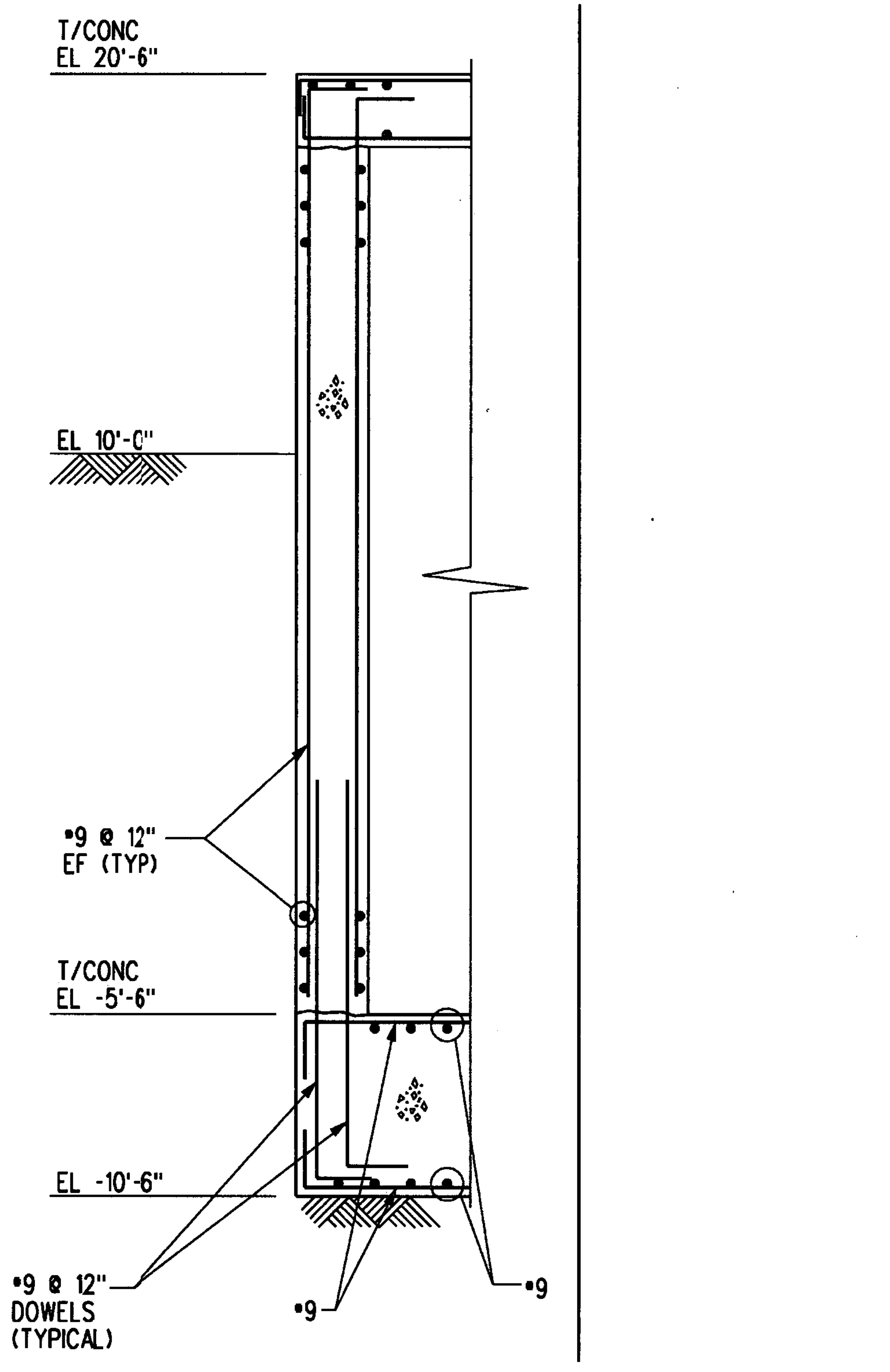


Figure 3E.4-7 {Reinforcement for UHS Electrical Building North Wall}



Enclosure 4

Markup of COLA Part 7,
Departures and Exemption Requests,

Calvert Cliffs Nuclear Power Plant, Unit 3

1.1 DEPARTURES

This Departure Report includes deviations in the CCNPP Unit 3 COL application FSAR from the information in the U.S. EPR FSAR, pursuant to 10 CFR Part 52. The U.S. EPR Design Certification Application is currently under review with the NRC. However, for the purposes of evaluating these deviations from the information in the U.S. EPR FSAR, the guidance provided in Regulatory Guide 1.206, Section C.IV.3.3, has been utilized.

The following Departures are described and evaluated in detail in this report:

1. Maximum Differential Settlement (across the basemat)
2. Maximum Annual Average Atmospheric Dispersion Factor (0.5 mile – limiting sector)
3. Accident Atmospheric Dispersion Factor (0-2 hour, Low Population Zone, 1.5 miles)
4. Toxic Gas Detection and Isolation
5. Shear Wave Velocity
6. In-Structure Response Spectra
7. Normal Power Supply System
8. Coefficient of Static Friction

1.1.1 MAXIMUM DIFFERENTIAL SETTLEMENT (ACROSS THE BASEMAT)

Affected U.S. EPR FSAR Sections: Tier 1 Table 5.0-1, Tier 2 Table 2.1-1, Tier 2 Section 2.5.4.10.2

Summary of Departure:

The U.S. EPR FSAR identifies a maximum differential settlement of 1/2 inch in 50 feet (i.e., 1/1200) in any direction across the basemat. The estimated settlement values for the Nuclear Island common basemat, Emergency Generating Building foundations, and Essential Service Water System Cooling Tower foundations exceed the U.S. EPR FSAR value.

Extent/Scope of Departure:

This Departure is identified in CCNPP Unit 3 FSAR Table 2.0-1 and Section 2.5.4.10.2.

Departure Justification:

The estimated site-specific values for settlement of the CCNPP Unit 3 Nuclear Island common basemat foundation are in the range of 1/600 (1 inch in 50 feet) to 1/1200 (1/2 inch in 50 feet) as stated in FSAR Section 2.5.4.10.2.

As described in FSAR Section 3.8.5.5.1, to account for the Calvert Cliffs site-specific expected differential settlement values, an evaluation of differential settlements up to 1/600 (1 inch in 50 feet) was performed. The evaluation consisted of a static finite element analysis of the foundation structures which considered the effects of the higher expected displacement (tilt) on the foundation bearing pressures and basemat stress due to structural eccentricities resulting from a uniform rotation of the foundation mat along the axis of the nuclear island common basemat. The evaluation assumed no changes in the soil stiffness or increased flexure due to differential settlement consistent with the design analysis for the standard U.S. EPR design. The evaluation considered Soil Case SC15, from the U.S. EPR FSAR standard design, which represented the softest soil condition used in the U.S. EPR standard plant design and exhibits the largest differential displacements of the basemat. Results from the evaluation indicate there is negligible difference in both the soil bearing pressures and the stresses in the concrete basemat structure when the Nuclear Island is subjected to an initial settlement of 1/600 (1 inch in 50 feet) as compared to the U.S. EPR standard plant analysis results that were based on an initial settlement of 1/1200 (1/2 inch in 50 feet). Therefore, the site specific departure in differential settlement values is structurally acceptable.

The estimated site-specific differential settlement for the Emergency Power Generating Buildings and Essential Service Water System Cooling Towers (based on a fully flexible basemat) are ~~1/550 and 1/600 (1 inch in 50 feet)~~ 1/1166 and 1/845 (approximately ½ inch and ¾ inch in 50 ft), respectively, as stated in FSAR Section 2.5.4.10.2.

As described in Sections FSAR 3.8.5.5.2 and 3.8.5.5.3, finite element analyses were performed for the Emergency Power Generating Buildings and Essential Service Water System Cooling Towers using soil springs representing the CCNPP Unit 3 site. For each structure, the differential settlement within the confines of the building periphery is shown to be substantially less than the 1/1200 (1/2 inch in 50 feet) requirement of the U.S. EPR FSAR.

The variation of the finite element analysis differential settlement with the estimated differential settlements of Section 2.5.4.10.2 is attributed to the conventional geotechnical treatment of the foundation as a flexible plate, a condition much more conservative than the actual heavily stiffened (by deep reinforced concrete walls) 6'-0" thick reinforced concrete Emergency Power Generating Building and Essential Service Water System Cooling Tower basemats.

Finite element analyses were also performed to evaluate the effects of overall Emergency Power Generating Building and Essential Service Water System Cooling Tower tilts of L/550 and L/600, respectively, where L is the least basemat dimension. For these analyses:

- Spring stiffnesses are adjusted to achieve a tilt of L/550,
- The elliptical distribution of soil springs is maintained,
- Soil spring stiffnesses along the basemat centerline (perpendicular to the direction of tilt) are retained, and
- Adjustment is made to all other springs as a function of the distance from the basemat centerline to the edges.

Bending moments from these finite element analyses confirm that an uncracked condition of the Emergency Power Generating Building and Essential Service Water System Cooling Tower basemats is maintained.

Departure Evaluation:

This Departure, associated with the maximum differential settlement of the Nuclear Island common basemat, the Emergency Power Generating Building foundations, and Essential Service Water System Cooling Tower foundations, has been evaluated and determined to not adversely affect the safety function of these structures. Accordingly, the Departure does not:

1. Result in more than a minimal increase in the frequency of occurrence of an accident previously evaluated in the plant-specific FSAR;
2. Result in more than a minimal increase in the likelihood of occurrence of a malfunction of a structure, system, or component (SSC) important to safety and previously evaluated in the plant-specific FSAR;
3. Result in more than a minimal increase in the consequences of an accident previously evaluated in the plant-specific FSAR;
4. Result in more than a minimal increase in the consequences of a malfunction of an SSC important to safety previously evaluated in the plant-specific FSAR;

5. Create a possibility for an accident of a different type than any evaluated previously in the plant-specific FSAR;

6. Create a possibility for a malfunction of an SSC important to safety with a different result than any evaluated previously in the plant-specific FSAR;

7. Result in a design basis limit for a fission product barrier as described in the plant specific FSAR being exceeded or altered; or

8. Result in a departure from a method of evaluation described in the plant-specific FSAR used in establishing the design bases or in the safety analyses.

This Departure does not affect resolution of a severe accident issue identified in the plant-specific FSAR.

Therefore, this Departure has no safety significance.

1.1.9 Coefficient of Static Friction

Affected U.S. EPR FSAR Sections: Tier 2 Table 2.1-1, Tier 2 Sections 2.5.4.2, 3.8.5.4.2, and 3.8.5.6.1

Summary of Departure:

The U.S. EPR FSAR identifies a minimum coefficient of static friction of 0.7 at the soil basemat interface. The geotechnical site investigation for CCNPP Unit 3 indicates coefficients of static friction between 0.35 and 0.45 for the underlying soil layers including structural fill, as shown in FSAR Table 2.5-58. Static friction coefficients for various sliding interfaces under the Nuclear Island common basemat, the Emergency Power Generating Building foundations, and the Essential Service Water Building foundations are reported in FSAR Table 3.8-1. All the aforementioned coefficients of static friction are less than the U.S. EPR FSAR value of 0.7.

Scope/Extent of Departure:

This Departure is identified in Part 2 FSAR, Section 3.8.5.5.

Departure Justification:

As described in FSAR Section 3.8.5.5, site-specific sliding stability evaluations are performed for the Nuclear Island Common Basemat Structures, the Emergency Power Generating Buildings (EPGBs), and the Essential Service Water Buildings (ESWBs) under site SSE loading. The governing factors of safety against sliding exceed the minimum allowable value of 1.1, as specified by NUREG 0800, Standard Review Plan 3.8.5, Structural Acceptance Criteria II.5. The factors of safety are reported in FSAR Table 3.8-4. Passive soil pressure is not utilized in these evaluations.

Therefore, the Nuclear Island Common Basemat Structures, the Emergency Power Generating Buildings, and the Essential Service Water Buildings are stable, despite the lower coefficients of static friction.

Departure Evaluation:

This Departure, associated with static coefficient of friction used for the Nuclear Island Common Basemat Structures foundations, the Emergency Power Generating Building foundations, and the Essential Service Water Building foundations, has been evaluated and determined to not affect the safety function of these structures. Accordingly, this Departure does not:

1. Result in more than a minimal increase in the frequency of occurrence of an accident previously evaluated in the plant-specific FSAR;
2. Result in more than a minimal increase in the likelihood of occurrence of malfunction of a structure, system, or component (SSC) important to safety and previously evaluated in the plant-specific FSAR;

3. Result in more than a minimal increase in the consequences of an accident previously evaluated in the plant-specific FSAR.

4. Result in more than a minimal increase in the consequences of a malfunction of an SSC important to safety previously evaluated in the plant-specific FSAR;

5. Create a possibility for an accident of a different type than any evaluated previously in the plant-specific FSAR;

6. Create a possibility for a malfunction of an SSC important to safety with a different result than any evaluated previously in the plant specific FSAR;

7. Result in a design basis limit for a fission product barrier as described in the plant-specific FSAR being exceeded or altered; or

8. Result in a departure from a method of evaluation described in the plant-specific FSAR used in establishing the design bases or in the safety analyses.

This Departure does not affect resolution of a severe accident issue identified in the plant-specific FSAR.

Therefore, this Departure has no safety significance.

1.2.1 MAXIMUM DIFFERENTIAL SETTLEMENT (ACROSS THE BASEMAT)

Applicable Regulation: 10 CFR Part 52

The U.S. EPR FSAR Tier 1 Table 5.0-1, Tier 2 Table 2.1-1, and Tier 2 Section 2.5.4.10.2 identify a maximum differential settlement of 1/2 inch in 50 feet (i.e., 1/1200) in any direction across the basemat. The estimated settlement values for the Nuclear Island common basemat, Emergency Generating Building foundations, and Essential Service Water System Cooling Tower foundations exceed the U.S. EPR FSAR value.

Pursuant to 10 CFR 52.7 and 10 CFR 52.93, Calvert Cliffs 3 Nuclear Project, LLC, and UniStar Nuclear Operating Services, LLC, request an exemption from compliance with the U.S. EPR FSAR Tier 1 and 2 requirements associated with the maximum differential settlement.

Discussion:

The estimated site-specific values for settlement of the CCNPP Unit 3 Nuclear Island common basemat foundation are in the range of 1/600 (1 inch in 50 feet) to 1/1200 (1/2 inch in 50 feet) as stated in FSAR Section 2.5.4.10.2.

As described in FSAR Section 3.8.5.5.1, an evaluation of differential settlements up to 1/600 (1 inch in 50 feet) was performed. The evaluation consisted of a static finite element analysis of the foundation structures which considered the effects of the higher expected displacement (tilt) on the foundation bearing pressures and basemat stress due to structural eccentricities resulting from a uniform rotation of the foundation mat along the axis of the nuclear island common basemat. The evaluation assumed no changes in the soil stiffness or increased flexure due to differential settlement consistent with the design analysis for the standard U.S. EPR design. The evaluation considered Soil Case SC15, from the U.S. EPR FSAR standard design, which represented the softest soil condition used in the U.S. EPR standard plant design and exhibits the largest differential displacements of the basemat. Results from the evaluation indicate there is negligible difference in both the soil bearing pressures and the stresses in the concrete basemat structure when the Nuclear Island is subjected to an initial settlement of 1/600 (1 inch in 50 feet) as compared to the U.S. EPR standard plant analysis results that were based on an initial settlement of 1/1200 (1/2 inch in 50 feet). Therefore, the site specific departure in differential settlement values is structurally acceptable.

The estimated site-specific differential settlement for the Emergency Power Generating Buildings and Essential Service Water System Cooling Towers (based on a fully flexible basemat) are ~~1/550 and 1/600 (1 inch in 50 feet)~~ 1/1166 and 1/845 (approximately ½ inch and ¾ inch in 50 ft), respectively, as stated in FSAR Section 2.5.4.10.2.

As described in Sections FSAR 3.8.5.5.2 and 3.8.5.5.3, finite element analyses were performed for the Emergency Power Generating Buildings and Essential Service Water System Cooling Towers using soil springs representing the CCNPP Unit 3 site. For each structure, the differential settlement within the confines of the building periphery is shown to be substantially less than the 1/1200 (1/2 inch in 50 feet) requirement of the U.S. EPR FSAR.

The variation of the finite element analysis differential settlement with the estimated differential settlements of Section 2.5.4.10.2 is attributed to the conventional geotechnical treatment of the foundation as a flexible plate, a condition much more conservative than the actual heavily stiffened (by deep reinforced concrete walls) 6'-0" thick reinforced concrete Emergency Power Generating Building and Essential Service Water System Cooling Tower basemats.

Finite element analyses were also performed to evaluate the effects of overall Emergency Power Generating Building and Essential Service Water System Cooling Tower tilts of L/550 and L/600, respectively, where L is the least basemat dimension. For these analyses:

- Spring stiffnesses are adjusted to achieve a tilt of L/550,
- The elliptical distribution of soil springs is maintained,
- Soil spring stiffnesses along the basemat centerline (perpendicular to the direction of tilt) are retained, and
- Adjustment is made to all other springs as a function of the distance from the basemat centerline to the edges.

Bending moments from these finite element analyses confirm that an uncracked condition of the Emergency Power Generating Building and Essential Service Water System Cooling Tower basemats is maintained.

This change associated with the maximum differential settlement of the Nuclear Island common basemat, the Emergency Power Generating Building foundations, and Essential Service Water System Cooling Tower foundations, has been evaluated and determined to not adversely affect the safety function of these structures. Therefore, this change will not result in a significant decrease in the level of safety otherwise provided by the design described in the U.S. EPR FSAR.

The exemption is not inconsistent with the Atomic Energy Act or any other statute. As such, the requested exemption is authorized by law.

This change does not result in a departure from the design and does not require a change in the design described in the U.S. EPR FSAR. In addition, the change has been evaluated and determined to not adversely affect the safety function of the associated structures. Therefore, the requested exemption will not present an undue risk to the public health and safety.

The change does not relate to security and does not otherwise pertain to the common defense and security. Therefore, the requested exemption will not endanger the common defense and security.

The special circumstance necessitating the request for exemption is that the CCNPP Unit 3 Nuclear Island common basemat, the Emergency Power Generating Building foundations, and Essential Service Water System Cooling Tower foundations estimated settlement values exceed the U.S. EPR FSAR value. However, the CCNPP Unit 3 specific maximum differential settlement of the Nuclear Island common basemat, the Emergency Power Generating Building foundations, and Essential Service Water System Cooling Tower foundations, has been evaluated and determined to not adversely affect the safety function of these structures. As such, application of the regulation for this particular circumstance would not serve the underlying purpose of the rule and is not required to achieve the underlying purpose of the rule.

This requested exemption does not require a change in the design described in the U.S. EPR FSAR. Therefore, this exemption will not result in any loss of standardization.

For these reasons, Calvert Cliffs 3 Nuclear Project, LLC, and UniStar Nuclear Operating Services, LLC, request approval of the requested exemption from compliance with the U.S. EPR FSAR Tier 1 and 2 requirements associated with maximum differential settlement.

Enclosure 5

Markup of COLA Part 10
"Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) and ITAAC Closure."

Calvert Cliffs Nuclear Power Plant, Unit 3

2.4 SITE-SPECIFIC ITAAC

The Site-Specific ITAAC are provided in {Table 2.4-1 through Table 2.4-~~31~~37}. Site-specific systems were evaluated against selection criteria in {CCNPP Unit 3} FSAR Section 14.3.

Part 10: ITAAC

Table 2.4-6—{Essential Service Water Buildings Inspections, Tests, Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Tests, or Analysis	Acceptance Criteria
2	For the Essential Service Water Buildings' below grade concrete foundations and walls, a low water to cement ratio concrete will be utilized.	Tests, inspections, or a combination of tests and inspections will be conducted to ensure the concrete meets the low water to cement ratio limit.	A report exists that concludes the concrete utilized to construct the as-built Essential Service Water Buildings' below grade concrete foundation and walls have a maximum water to cementitious materials ratio of ~~0.45.~~0.40.

Part 10: ITAAC

Table 2.4-7—{Ultimate Heat Sink Makeup Water Intake Structure Inspections, Tests, Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Tests, or Analysis	Acceptance Criteria
3	For the UHS Makeup Water Intake Structure's below grade concrete foundation and walls, a low water to cement ratio concrete mixture will be utilized.	Tests, inspections, or a combination of tests and inspections will be conducted to ensure the concrete meets the low water to cement ratio limit.	A report exists that concludes the concrete utilized to construct the as-built UHS Makeup Water Intake Structure's below grade concrete foundation and walls have a maximum water to cementitious materials ratio of ~~0.45.~~ 0.40.

Table 2.4-8—{Ultimate Heat Sink Electrical Building Inspections, Tests, Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Tests, or Analysis	Acceptance Criteria
3	For the UHS Electrical Building's below grade concrete foundation and walls, a low water to cement ratio concrete mixture will be is utilized.	Tests, inspections, or a combination of tests and inspections will be conducted to ensure the concrete meets the low water to cement ratio limit.	A report exists that concludes the concrete utilized to construct the as-built UHS Electrical Building's below grade concrete foundation and walls have a maximum water to cementitious materials ratio of ~~0.45.~~ 0.40.

Table 2.4-9—{Buried Duct Banks and Pipes Inspections, Tests, Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Tests, or Analysis	Acceptance Criteria
7	For the concrete components of buried Seismic Category I electrical duct banks and pipes, a low water to cement ratio concrete mixture will be is utilized.	Tests, inspections, or a combination of tests and inspections will be conducted to ensure the concrete meets the low water to cement ratio limit.	A report exists that concludes the concrete utilized to construct the concrete components of as-built buried Seismic Category I electrical duct banks and pipes have a maximum water to cementitious materials-ratio of ~~0.45.~~ 0.40.

Table 2.4-19—{Circulating Water Makeup Intake Structure Inspections, Tests,Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Tests, or Analysis	Acceptance Criteria
2	For the Circulating Water Makeup Intake Structure's below grade concrete foundation and walls, a low water to cement ratio concrete mixture will be is utilized.	Tests, inspections, or a combination of tests and inspections will be conducted to ensure the concrete meets the low water to cement ratio limit.	A report exists that concludes the concrete utilized to construct the as-built Circulating Water Makeup Intake Structure's below grade concrete foundation and walls have a maximum water to cementitious materials-ratio of ~~0.45.~~ 0.40.

Table 2.4-33-{Forebay Structure Inspections, Tests, Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Tests, or Analysis	Acceptance Criteria
2	For the Forebay Structure below grade concrete foundation and walls, a low water to cement ratio concrete mixture will be is utilized.	Tests will be conducted to ensure the concrete meets the low water to cement ratio limit.	A report exists that concludes the concrete utilized to construct the as-built Forebay Structure below grade concrete foundation and walls have a maximum water to cementitious materials-ratio of ~~0.45.~~ 0.40.

Table 2.4-37-{Waterproofing Geomembrane Under Nuclear Island Common Basemat Structures and Other Buildings Inspections, Tests, Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Tests, or Analysis	Acceptance Criteria
1	Coefficient of static friction at the horizontal interface of HDPE geomembrane and sand is greater than or equal to 0.52.	Laboratory testing will be performed in accordance with ASTM D5321 and/or ASTM D6467 to verify the design coefficient of static friction at the horizontal interface of HDPE geomembrane and sand.	A report exists that concludes the coefficient of static friction at the horizontal interface of HDPE geomembrane and sand is greater than or equal to 0.52.

Enclosure 6

Conforming Changes to other FSAR Sections and COLA Parts

Calvert Cliffs Nuclear Power Plant, Unit 3

1.8.2 DEPARTURES

The U.S. EPR FSAR includes the following COL Item in Section 1.8.2:

> A COL applicant that references the U. S. EPR design certification will provide a list of any departures from the FSAR in the COL FSAR.

This COL Item is addressed as follows:

{The list of departures from the U.S. EPR FSAR is as follows:

Maximum Differential Settlement	FSAR 2.5.4 and 3.8.5
Maximum Annual Average Atmospheric Dispersion Factor	FSAR 2.3.5
Accident Atmospheric Dispersion Factor from 0 - 2 Hours for the Low Population Zone	FSAR 2.3.4 and 15.0.3
In-Structure Response Spectra	FSAR 3.7.2.5.2
Toxic Gas Detection and Isolation	FSAR 3.11, 6.4, 9.4.1 and 14.2.12
Technical Specifications Setpoint Control Program	FSAR 16.3.3, 16.5.5, and Bases 16.3.3
Coefficient of Static Friction	FSAR 3.8.5.5

Justification for these departures is presented in Part 7 of the COL application.}

Table 2.0-1—{U.S. EPR Site Design Envelope Comparison}
(Page 2 of 5)

	U.S. EPR FSAR Design Parameter Value/Characteristic	CCNPP Unit 3 Design Parameter Value/Characteristic
	Flood Level	
Maximum Flood (or Tsunami)	1 ft below grade	Approximately 3 ft below grade, except for the Forebay, UHS Makeup Water Intake Structure and UHS Electrical Building which is designed to function under submerged conditions (See Sections 2.4.1 and 2.4.2, 2.4.10, 3.4.2, 3.4.3.10, 3.8.4.1.11, 3.8.4.3, and 9.2.5)
	Wind	

3.7.2.14.2 EPGB, ESWB, Common Basemat Intake Structures and UHS Electrical Building

The stability of the EPGB, ESWB, CBIS and the UHS Electrical Building for seismic loading is determined using the stability load combinations provided in NUREG-0800 Section 3.8.5, Acceptance Criteria 3 (NRC, 2007a)~~, listed as Load Combination 7 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 response time histories of reactions at the basemat nodes. These responses include the effects of seismic forces, static and dynamic lateral earth pressures, and hydrostatic and hydrodynamic forces. The following steps are used to assess the seismic stability:

Table 14.3-2—{Site Specific SSC ITAAC Screening Summary}

Site-Specific Structure, System, or Component	U.S. EPR Interface	Selected for ITAAC
Component		
Buried Ductbanks	Yes	Yes
Buried Pipe	Yes	Yes
New and Spent Fuel Storage Racks	Yes	Yes
Waterproofing Membrane for Seismic Category I ~~and Seismic Category II-SSE~~ Concrete Foundations ~~and Walls~~, Walls and Buried Concrete Commodities Exposed to Low pH Groundwater	No	Yes
System		