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UniStar
NUCLEAR ENERGY

10 CFR 50.4
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June 28, 2013

UN#13-086

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

Subject: UniStar Nuclear Energy, NRC Docket No. 52-016
Partial Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3,
RAI No. 315, Seismic System Analysis

References:
1) Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 315 SEB2 5927," dated August 3, 2011

2) UniStar Nuclear Energy Letter UN#13-006, from Mark T. Finley to Document Control Desk, U.S. NRC, Updated RAI Closure Plan, dated January 30, 2013

The purpose of this letter is to provide a partial response to the request for additional information (RAI) identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated August 3, 2011 (Reference 1). This RAI addresses Seismic System Analysis, as discussed in Section 3.7 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 9.

Reference 2 indicated that a response to RAI 315, Question 03.07.02-63 would be provided to the NRC by June 29, 2013. Enclosure 1 provides our response to RAI 315, Question 03.07.02-63 regarding the Turbine Building and Switchgear Building, which are both located on the Turbine Island Structure, and includes revised COLA content. The response regarding the Access Building (AB) will be provided together with the response to RAI 315, Question

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03.07.02-64 for the Nuclear Island (NI) per the schedule indicated in Reference 2, because the AB is located on the NI.

Enclosure 2 provides the COLA impact of the response to RAI 315, Question 03.07.02-63. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

Enclosure 3 provides a table of changes to the CCNPP Unit 3 COLA associated with the RAI 315, Question 03.07.02-63 response. As identified in the Enclosure 3 Table of Changes, this response modifies previously submitted RAI responses.

Our response does not include any new regulatory commitments. This letter, and its enclosures, does not contain any sensitive or proprietary information.

If there are any questions regarding this transmittal, please contact me at (410) 369-1907 or Mr. Wayne A. Massie at (410) 369-1910.

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

Executed on June 28, 2013

Mark T. Finley

Enclosures:
1) Response to NRC Request for Additional Information RAI 315, Question 03.07.02-63, Seismic System Analysis, Calvert Cliffs Nuclear Power Plant, Unit 3
2) Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 315, Question 03.07.02-63, Calvert Cliffs Nuclear Power Plant, Unit 3
3) Table of Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 315, Question 03.07.02-63, Calvert Cliffs Nuclear Power Plant, Unit 3

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
John Fringer, NRC Environmental Project Manager, U.S. EPR COL Application
Amy Snyder, NRC Project Manager, U.S. EPR DC Application, (w/o enclosures)
Patricia Holahan, Acting Deputy Regional Administrator, NRC Region II, (w/o enclosures)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2,
David Lew, Deputy Regional Administrator, NRC Region I (w/o enclosures)

Enclosure 1

Response to NRC Request for Additional Information, RAI 315, Question 03.07.02-63 Seismic System Analysis, Calvert Cliffs Nuclear Power Plant, Unit 3

RAI No. 315

Question 03.07.02-63

Follow-up Question to RAI 253, Question 03.07.02-46

In its partial response to Question 03.07.02-46, the applicant stated that the TB, SB, and AB will be analyzed and designed to prevent their failure under site-specific SSE loading conditions with a margin of safety equivalent to that of Seismic Category I structures. In addition the applicant states that to assure there is no seismic interaction between these Seismic Category II structures and the nearest Seismic Category I structure it will be verified that the separation distances are greater than their combined elastic deformations. However, for the staff to complete its review following the guidance provided in SRP 3.7.2, and be able to conclude that seismic interaction will not occur between the Seismic Category I structures and site-specific Category II structures, the applicant is requested to provide the seismic input, assumptions, modeling techniques, and methods of dynamic analysis that are used to determine the seismic loads required for design of the Seismic Category II structures. In addition, the applicant is requested to describe how these structures are designed to have a margin of safety equivalent to that of category I structures, and how the displacements for these structures are determined so as to verify the separation distances between the Seismic Category I structures and site-specific Category II structures are adequate. The applicant is also requested to provide the methods used to determine the seismic stability of these structures and to provide the results including the factors of safety against sliding and overturning. Also, include site-specific ITAAC for each structure to confirm that the as-built structure is analyzed and designed as described in the FSAR.

Response

For clarity of response, the question is divided into the following parts:

1. Provide the seismic input, assumptions, modeling techniques, and methods of dynamic analysis that are used to determine the seismic loads required for design of the Seismic Category II structures.
2. Describe how these structures are designed to have a margin of safety equivalent to that of Seismic Category I structures, and how the displacements for these structures are determined, so as to verify that the separation distances between the Seismic Category I structures and site-specific Seismic Category II structures are adequate.
3. Provide the methods used to determine the seismic stability of these structures and to provide the results, including the factors of safety against sliding and overturning.
4. Provide site-specific Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) for each structure to confirm that the as-built structure is analyzed and designed as described in the Final Safety Analysis Report (FSAR).

The site-specific Seismic Category (SC) II Turbine Building (TB) and Switchgear Building (SB) are contiguous with each other, and are analyzed and designed as one structure called the Turbine Island Structure (TI Structure). The TI Structure is described in the COLA FSAR Subsection 3.7.2.3.3 and is classified as a nonsafety-related Augmented Quality (NS-AQ) structure in Combined License Application (COLA) FSAR Table 3.2-1.

The Access Building (AB) is physically located on the Nuclear Island (NI), and is evaluated with the NI.

1. Seismic parameters of the TI Structure, including design ground motion, design ground motion response spectra, and design ground motion time history, are based on the site-specific safe shutdown earthquake (SSE). Associated changes to the COLA FSAR Subsections 3.7.1.1 (design ground motion), 3.7.1.1.1.5 (design ground motion response spectra), and 3.7.1.1.2.4 (design ground motion time history) have been made. The TI Structure, along with assumptions, is described in COLA FSAR Subsection 3.7.2.3.3. Modeling techniques for the SSI analysis are provided in COLA FSAR Subsection 3.7.2.4. Structural steel and concrete elements of the TI Structure are preliminary and shall be finalized after equipment weights, sizes and configuration are confirmed. Description of the supporting media is provided in Subsection 3.7.1.3.4. Complex Frequency Response Analysis Method, using a finite element model of the TI Structure, is used for the Soil Structure Interaction (SSI) analysis, as described in COLA FSAR Subsections 3.7.2.1.3. The SSI analysis input motions for the TI structure are developed based on the 2012 Central and Eastern United States Seismic Source Characterization (CEUS SSC) model. These motions are amplified by the applicable structure-to-soil-to-structure interaction (SSSI) factors, due to the SSSI effects of the adjacent SC I structures.

2. The TI Structure is conservatively analyzed and designed as a SC I structure, for site-specific SSE loads. Loads, load combinations, and the allowable stresses for the design of concrete elements of the SC II TI Structure are per the requirements of ACI 349. Loads, load combinations, and the allowable stresses for the design of steel components of the TI Structure are in accordance with the requirements of ANSI/AISC N690. The TI Structure is thus designed to stay elastic and meet the NUREG-0800 Section 3.7.2 acceptance criteria 8C. The closest SC I structure to the TI Structure is the Nuclear Island Common Basemat Structure. Minimum separation distance between the SC I Nuclear Island Common Basemat Structure and the SC II TI Structure is approximately 30 feet, as described in the COLA FSAR Subsection 3.7.2.8 and as shown in Figure 3B-1 of U.S. EPR FSAR. This separation is sufficiently wide, such that there is no possibility of interaction between the SC I Nuclear Island Common Basemat Structure and the SC II TI Structure. During detailed design phase, deflections will be calculated for applicable load combinations and the sum of deflections of the SC I Nuclear Island Common Basemat Structure and the SC II TI Structure verified to be less than the separation distance between these two structures.

3. The SSI analysis is performed for Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 site-specific SSE loads to determine the stability of the TI Structure against overturning and sliding failures. Details are provided in COLA FSAR Sections 3.7.1, 3.7.2, and in the new Figures 3.7-82 through 3.7-86. The results are documented in COLA FSAR Table 3.8-4. These factors of safety are within the requirements of Standard Review Plan (SRP) 3.8.5. The analysis of the TI structure is based on the conceptual structural layout of the elements and associated equipment loading. The Factor of Safety (FOS) for sliding and overturning analysis will be reconfirmed to the requirements of SRP 3.8.5 during the detailed design when detailed structural members sizing information will be developed.

4. Tables 2.4-10 and 2.4-11 in COLA Part 10, ITAAC, have been updated to verify that the site-specific TI Structure separation distance is greater than design analysis.

COLA Impact

Enclosure 2 provides the COLA impact of the response to RAI 315, Question 03.07.02-63.

Enclosure 2

Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 315, Question 03.07.02-63, Calvert Cliffs Nuclear Power Plant, Unit 3

FSAR Subsection 3.3.1.2, using the site-specific 100-year recurrence interval wind speed. The velocity pressure exposure coefficient (K_z), topographic factor (K_{zt}), and wind directionality factor (K_d) are determined in conformance with ASCE 7-05 (ASCE, 2005) for the site-specific conditions, and an importance factor of 1.15 is used.}

3.3.2 Tornado Loadings

The U.S. EPR FSAR includes the following COL Item in Section 3.3.2:

> A COL applicant that references the U.S. EPR design certification will demonstrate that failure of site-specific structures or components not included in the U.S. EPR standard plant design, and not designed for tornado loads, will not affect the ability of other structures to perform their intended safety functions.

This COL Item is addressed as follows:

{A discussion of site-specific structures not designed for tornado loadings is provided in Section 3.3.2.3.}

3.3.2.1 Applicable Tornado Design Parameters

{No departures or supplements.}

3.3.2.2 Determination of Tornado Forces on Structures

No departures or supplements.

3.3.2.3 Interaction of Non-Seismic Category I Structures with Seismic Category I Structures

{Non-safety-related structures located on the site and not included in U.S. EPR FSAR Section 3.3.2.3 include:

- Fire Protection Water Tanks
- Fire Protection Building
- Storage / Warehouse
- Central Gas Supply Building
- Security Access Facility
- Switchgear Building
- Grid Systems Control Building
- Circulating Water System Cooling Tower
- Circulating Water System Pump Building
- Circulating Water System Makeup Water Intake Structure
- Circulating Water System Retention Basin
- Desalinization/Water Treatment Plant

- Waste Water Treatment Facility
- Sheet Pile Wall
- Demineralized Water Tanks

Except for the Switchgear Building, Sheet Pile Wall, and Circulating Water System (CWS) Makeup Water Intake Structure (MWIS), the non-safety-related buildings are miscellaneous steel and concrete structures, which are not designed for tornado loadings. These structures are distant enough from safety-related structures that their collapse due to tornado loadings would not result in adverse interaction with any safety-related structure. During detailed design of such structures, their heights and separation distances from safety-related structures will be maintained such that the failure of these structures due to tornado loadings will not affect the ability of safety-related structures to perform their intended safety functions. Missiles generated by the collapse of these structures during tornado loadings are enveloped by the design basis tornado missile loads described in U.S. EPR FSAR Section 3.5.1.4.

TI Structure

the

~~The Switchgear Building~~, Sheet Pile Wall, and CWS MWIS have potential for interaction with safety-related structures and are designed to withstand the effects of tornado loadings as described below:

TI Structure

The ~~structural system of the Switchgear Building~~ employs engineered pressure relief sliding panels to mitigate the effects of tornado loadings. Potential missiles generated by detachment of these siding panels are addressed in Subsection 3.5.1.4.

The layout of the Sheet Pile Wall and the separation distance between the Sheet Pile Wall and the nearest Seismic Category (SC) I structure, system, and component (SSC) will preclude any interaction between the Sheet Pile Wall and the SC I SSC for tornado loads.

The reinforced concrete portion of the CWS MWIS is designed for tornado loadings. Should collapse of the aboveground steel structure occur, it cannot directly impact any SC I SSC. Since the reinforced concrete portion supporting the steel structure is integrally connected to SC I Forebay Structure, the reinforced concrete portion is analyzed to demonstrate that the collapse of the steel structure due to tornado loads does not impair the integrity of SC I SSCs.

3.3.3 References

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

3.7 SEISMIC DESIGN

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

3.7.1 Seismic Design Parameters

{Section 3.7.1 and Appendix 3F describe the site-specific seismic design characteristics for CCNPP Unit 3. Section 3.7.2 provides the methodology and results of the confirmatory site-specific Soil-Structure Interaction (SSI) analysis. The confirmatory analysis of the Nuclear Island (NI) is based on:

- a surface mounted structure

- the in-situ soil SWV profile, without the backfill as described in Section 3.8.4.6.1, and

- the site-specific SSE response spectra.

The results of this confirmatory analysis are not used for design because the US EPR Design of the NI, EPGB, and ESWB are adopted for CCNPP3. Section 2.5.2.6 compares the site-specific seismic characteristics and the results of the confirmatory analysis with the US EPR Analysis and Design. This comparison confirms that the US EPR seismic design envelopes the CCNPP Unit 3 site by a large margin. In addition, the SSI analysis of the site-specific Seismic Category I structures, listed below, is presented in Section 3.7.2.

Throughout this section, three groups of structures are considered:

- Nuclear Island (NI) Common Basemat Structures
- Emergency Power Generating Buildings (EPGB) and Essential Service Water Buildings (ESWB) located in the NI area
- Site-specific Seismic Category I structures

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

- Ultimate Heat Sink (UHS) Makeup Water Intake Structure
- Forebay
- Buried Electrical Duct Banks and Pipes

The site-specific Seismic Category II structure, analyzed and designed as Seismic Category I, is:
♦ The Turbine Island (TI) Structure (i.e., combined Turbine Building and Switchgear Building)

The Turbine Island (TI) Structure is a Seismic Category II structure analyzed and designed for the same margin of safety as Seismic Category I structures to preclude any interaction with adjacent Safety Related Seismic Category I Structures (i.e., the ESWB, EPGB and Safeguard Buildings). Figure 3.8-1 shows the location of the TI Structure.

Two site-specific Seismic Category I structures: the UHS Makeup Water Intake Structure and the UHS Forebay, as well as the Seismic Category II Circulating Water Makeup Water Intake Structure share the same basemat; they are referred to as Common Basemat Intake Structures (CBIS). The CBIS are situated at the CCNPP Unit 3 site along the west bank of the Chesapeake Bay. Figures 9.2-4, 9.2-5 and 9.2-6 provide plan views of the Seismic Category I UHS structures, along with associated sections. Figures 10.4-4 and 10.4-5 provide the plan and section views of the Seismic Category II Circulating Water Makeup Intake Structure. The bottom of the CBIS basemat is situated approximately 37.5 ft (11.4 m) below a nominal grade elevation of 10 ft (3.0 m) NGVD 29. The layout of the Seismic Category I buried electrical duct banks and Seismic Category I buried piping is defined in Figures 3.8-1 and 3.8-2, and Figures 3.8-3 and 3.8-4, respectively.

In the SSI analysis, the time histories are applied at the FIRS horizon as "within" motions and are used in conjunction with the respective SSI soil profiles, described in Section 3.7.1.3.2.

3.7.1.1.2.3 Design Ground Motion Time History for Common Basemat Intake Structures

In the case of the CBIS, which are analyzed as embedded structures, the "within" acceleration time histories at each FIRS horizon are calculated using the computer program SHAKE2000 (described in Appendix 3F). In this analysis, the Site SSE spectrally matched time histories are used as input "outcrop" motions at the foundation level in conjunction with the strain-compatible profiles for the Intake area, presented in Section 3.7.1.3.3. No further iterations on soil properties are performed as the acceleration time history is converted from "outcrop" to "within." The analysis results in a set of three "within" motions (two horizontal and one vertical) at the same FIRS horizon. Three sets are developed corresponding to the LB, BE and UB profiles for the CBIS, as presented in Figure 3.7-16 through Figure 3.7-18. The development of the within acceleration time histories is discussed in detail in Appendix 3F. The time histories are applied at the FIRS horizon as "within" motions and are used in conjunction with the corresponding SSI soil profiles, described in Section 3.7.1.3.3.

3.7.1.2 Percentage of Critical Damping Values

Operating Basis Earthquake (OBE) structural damping values, defined in Table 2 of RG 1.61, Rev 1 (NRC, 2007c), are used for the dynamic analysis of site-specific Seismic Category I SSCs and confirmatory SSI analysis of the NI Common Basemat Structures as well as for the EPGB and ESWB. In-structure response spectra (ISRS) for site-specific Seismic Category I structures are also based on OBE structural damping values.

For the Turbine Island Structure, the SSE damping value for bolted steel structures from Table 1 of RG 1.61, Rev 1 (NRC, 2007c) is used.

The damping values for site-specific Seismic Category II structures are in accordance with RG 1.61, Rev. 1 (NRC, 2007c).

3.7.1.3 Supporting Media for Seismic Category I Structures

3.7.1.3.1 Nuclear Island Common Basemat

The supporting media for the seismic analysis of the NI Common Basemat Structures is shown in Figure 3.7-19 and Table 3.7-2 through Table 3.7-4. The presented soil profiles are site-specific and are strain-compatible with the Site SSE. Lower bound and upper bound profiles are calculated maintaining a minimum variation of 0.5 on the shear modulus. An evaluation of the CCNPP Unit 3 site-specific soil profiles with respect to the criteria provided in U.S. EPR FSAR Section 2.5.2.6 is described in Section 2.5.2.6.

Confirmatory site-specific SSI analyses are performed, as described in Section 3.7.2. The resulting in-structure response spectra (ISRS) at representative locations of the NI structures, as reported in Section 3.7.2.5.1, are found to be bounded by the corresponding U.S. EPR FSAR ISRS.

3.7.1.3.2 EPGB and ESWB

The supporting media for the seismic analysis of the EPGB and ESWB in the NI area are presented in Figure 3.7-21. The presented soil profiles are site-specific and are strain-compatible with the Site SSE. The development of the Site SSE strain-compatible soil profiles is described in detail in Appendix 3F.

Note that in contrast to Figure 3.7-19, where the top layer is located at the bottom of the NI common basemat foundation at approximately 40 ft (12 m) below grade, Figure 3.7-21

presents the profiles for the upper 656 ft (200m) with the top layer at grade, including the structural backfill layers, therefore consistent with the confirmatory SSI analyses of the EPGB and ESWB, described in Section 3.7.2.

3.7.1.3.4 Turbine Island Structure
The subsurface conditions under the TI Structure are equivalent to those used for the EPGB and ESWB. Therefore the soil property profiles, strain-compatible with Site SSE, presented in Section 3.7.1.3.2, are used in the SSI analysis of the TI Structure.

3.7.1.3.3 Common Basemat Intake Structures

The supporting media for the seismic analysis of the CBIS in the Intake area are presented in Figure 3.7-22 for the upper 656 ft (200m). The presented soil profiles are site-specific and are strain-compatible with the Site SSE. The development of the Site SSE strain-compatible soil profiles is described in detail in Appendix 3F. The dimensions of the CBIS, including the structural height, are described in Section 3.7.2.3.2.

3.7.1.4 References

CFR, 2008. Domestic Licensing of Production and Utilization Facilities, 10 CFR Part 50, U.S. Nuclear Regulatory Commission, February 2008.

McGuire, R.K., W.J. Silva, and C.J. Constantino, 2001. Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard and Risk-Consistent Ground Motion Spectra Guidelines, NUREG CR-6728, October, 2001.

Nuclear Energy Institute [NEI], 2009. Consistent Site-Response/Soil Structure Interaction Analysis and Evaluation. NEI White Paper, June 12, 2009 (ADAMS Accession No. ML091680715).

NRC, 1973. Design Response Spectra for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.60, Revision 1, U.S. Nuclear Regulatory Commission, December 1973.

NRC, 2007a. A Performance-Based Approach to Define the Site Specific Earthquake Ground Motion, Regulatory Guide 1.208, Revision 0, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2007b. Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants, NUREG-0800, Revision 3, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2007c. Damping Values for Seismic Design of Nuclear Power Plants, Regulatory Guide 1.61, Revision 1, U.S. Nuclear Regulatory Commission, March 2007.

NRC, 2009. Interim Staff Guidance on Ensuring Hazard-Consistent Seismic Input for Site Response and Soil Structure Interaction Analyses, DC/COL-ISG-017 Draft Issued for Comments.}

3.7.2 Seismic System Analysis

The U.S. EPR FSAR includes the following COL Item in Section 3.7.2:

> A COL applicant that references the U.S. EPR design certification will confirm that the site-specific seismic response is within the parameters of Section 3.7 of the U.S. EPR standard design.

This COL Item is addressed as follows:

{The confirmatory soil-structure interaction (SSI) analyses of Nuclear Island (NI) Common Basemat Structures, Emergency Power Generating Buildings (EPGBs) and Essential Service Water Buildings (ESWBs) for Site SSE and site-specific strain-compatible soil properties is addressed in Section 3.7.2.4.

Site-specific Seismic Category I structures at CCNPP Unit 3 include:

- Ultimate Heat Sink (UHS) Makeup Water Intake Structure (MWIS)
- Forebay

The Seismic Category I UHS Makeup Water Intake Structure and Seismic Category I Forebay are situated at the CCNPP Unit 3 site along the west bank of the Chesapeake Bay. These structures are part of the UHS Makeup Water System, which provides makeup water to the Essential Service Water Buildings for maintaining the safe shutdown of the plant 72 hours after a design basis accident. The UHS Makeup Water Intake Structure and Forebay are supported on a common basemat, which also supports the Seismic Category II Circulating Water Makeup Intake Structure. The UHS Makeup Water Intake Structure, Forebay, and Circulating Water Makeup Intake Structure, henceforth referred to as the Common Basemat Intake Structures (CBIS) in Section 3.7.2, are integrally connected. The Circulating Water Makeup Intake Structure and the UHS Makeup Water Intake Structure, respectively, are located on the north and south end of the Forebay. Figure 2.1-1 depicts the CCNPP Unit 3 site plan, which shows the position of the UHS Makeup Water Intake Structure and Forebay relative to the NI.

The bottom of the CBIS common basemat is situated approximately 37.5 ft (11.4 m) below a nominal grade elevation of 10 ft (3.0 m). 9.2-4, 9.2-5, and 9.2-6 provide plan views of the Seismic Category I structures, along with associated sections and details. 10.4-4 and 10.4-5 provide the plan and section views of the Seismic Category II Circulating Water Makeup Intake Structure.

As described in Section 3.7.2.3.3, an integrated finite element model is developed for the TI Structure. The complex frequency response analysis method is used for the seismic Soil Structure Interaction (SSI) analysis of the structure, with earthquake motion considered in three orthogonal directions (two horizontal and one vertical), as described in Section 3.7.2.6. The SSI analysis of site specific structures is performed, as described in Section 3.7.2.4, using Bechtel computer code SASSI2010, Version 1.1.

3.7.2.1 Seismic Analysis Methods

No departures or supplements.

3.7.2.1.1 Time History Analysis Method

No departures or supplements.

3.7.2.1.2 Response Spectrum Method

No departures or supplements.

3.7.2.1.3 Complex Frequency Response Analysis Method

As described in Section 3.7.2.3.2, an integrated finite element model is developed for the CBIS. The complex frequency response analysis method is used for the seismic SSI analysis of these structures, with earthquake motion considered in three orthogonal directions (two horizontal and one vertical) as described in Section 3.7.2.6. The SSI analysis of site-specific structures is performed, as described in Section 3.7.2.4, using RIZZO computer code SASSI, Version 1.3a. The hydrodynamic load effects are considered as described in Section 3.7.2.3.2.

3.7.2.1.4 Equivalent Static Load Method of Analysis

No departures or supplements.

3.7.2.2 Natural Frequencies and Response Loads

3.7.2.2.1 Nuclear Island Common Basemat Structures

Section 3.7.2.5.1 provides the in-structure response spectra (ISRS) for NI Common Basemat Structures for site-specific strain-compatible soil properties and Site SSE.

3.7.2.2.2 EPGB and ESWB

Section 3.7.2.5.2 provides the ISRS for EPGB and ESWB at the locations defined in U.S. EPR FSAR Section 3.7.2.5 for site-specific strain-compatible soil properties and Site SSE. Section 3.7.2.4.6.2 provides the combined average maximum nodal accelerations for the site-specific confirmatory SSI analysis.

3.7.2.2.3 Common Basemat Intake Structures

The SSI analysis of site-specific Seismic Category I structures is performed using the complex frequency response analysis method described in Section 3.7.2.1.3, where the equation of motion is solved in the frequency domain. The natural frequencies and associated modal analysis results are not obtained from this analysis. However, fixed base undamped eigenvalue analyses have been performed separately for the Common Basemat Intake Structures. The analysis results are tabulated in Table 3.7-5 and Table 3.7-6 for reference purposes only.

3.7.2.2.4 Turbine Island Structure
Section 3.7.2.8 presents the evaluation of the TI structure for seismic stability (i.e., overturning and sliding) based on results from the SSI dynamic analysis when subjected to the SSE motion, and using the corresponding strain-compatible profiles as supporting media.

Section 3.7.2.5.3 provides the ISRS at the locations of safety-related UHS Makeup Water pumps and facilities in the UHS Makeup Water Intake Structure at El. 11.5 ft and El. -22.5 ft, at the locations of safety-related traveling screens at EL. 21.0 ft, and at the location of safety-related electrical equipment at El. 26.5 ft. Section 3.7.2.4.6.3 provides the combined maximum nodal accelerations for the CBIS.

3.7.2.3 Procedures Used for Analytical Modeling

No departures or supplements.

3.7.2.3.1 Seismic Category I Structures – Nuclear Island Common Basemat

No departures or supplements.

3.7.2.3.2 Seismic Category I Structures – Not on Nuclear Island Common Basemat

As described in Section 3.7.2.4.2.2, the confirmatory SSI analysis of EPGB and ESWB is performed using finite element models.

The UHS Makeup Water Intake Structure and Forebay are the site-specific Seismic Category I structures situated away from the NI in the intake area.

The CBIS, i.e., the UHS Makeup Water Intake Structure, Forebay, and Circulating Water Makeup Intake Structure are reinforced concrete shear wall structures, and are supported on a 5 ft (1.5 m) thick reinforced concrete basemat. The Common Basemat Intake Structures extend approximately 260 ft (79.3 m) along the North-South direction and 89 ft (27.1 m) along the East-West direction, with respect to CCNPP Unit 3 coordinate system. The maximum height of the structures from the bottom of common basemat to the top of the UHS Makeup Water Intake Structure roof is approximately 69 ft (21.0 m).

Figures 9.2-4 through 9.2-6 and 10.4-4 and 10.4-5 are used as the bases for the development of the analytical model of the aforementioned structures.

A 3D finite element model of the CBIS is developed in STAAD Pro, Version 8i, as shown in Figures 3.7-23 and 3.7-24. The model is used to generate the finite element model for seismic SSI analysis using RIZZO computer code SASSI, Version 1.3a, and to perform static analysis for non-seismic loads.

The CBIS are symmetric about the North-South axis, as depicted in Figures 9.2-4 through 9.2-6 and 10.4-4 and 10.4-5. A sensitivity analysis was performed to consider the effects of the non-symmetric features such as door openings and equipment masses. Based on the sensitivity analysis, only one-half (western half) of the CBIS is modeled for the SSI analysis. Figure 3.7-23 depicts the finite element mesh for the half model.

The reinforced concrete basemat, floor slabs, and walls of the Common Basemat Intake Structures are modeled using plate/shell elements to accurately represent the structural geometry and to capture both in-plane and out-of-plane effects from applied loads. The finite element mesh is sufficiently refined to accurately represent the global and local modes of vibration. The finite element model in SASSI uses a thin shell element formulation that represents the in-plane and out-of-plane bending effects. In-plane shear deformation is accurately reproduced by the finite element mesh, while out-of-plane shear deformations are considered negligible due to the low thickness/height ratio of these walls.

The reinforced concrete basemat, floor slabs, and walls of the CBIS are modeled using thin shell elements in RIZZO computer code SASSI, Version 1.3a, to accurately represent the structural geometry and to capture in-plane membrane and out-of-plane bending. The average mesh size used in the finite element model below ground level and along the vertical direction is approximately 1.6 ft (0.5 m), based on one-fifth of the wave length at the highest frequency of the SASSI analysis. The average mesh size in the plan direction is approximately 5 ft (1.5 m), abased on an aspect ratio of approximately 3.0.

The skimmer walls, at the entrance of the UHS Makeup Water Intake Structure and the Circulating Water Makeup Intake Structure into the Forebay, have an inclination of approximately 10 degrees with the vertical. However, these walls are modeled vertically for simplification of the finite element model. This simplification has an insignificant effect on the global mass and stiffness distribution, and on the local responses of the structural panels.

The east and west bottom walls of the Forebay, to the top portion of the forebay wall corners, and the basemat below the backfill inside the UHS MWIS are the only structural panels that will crack during any of the applicable loading conditions. These walls crack since they retain approximately 37.5 ft (11.5 m) of soil and exhibit cantilever behavior. The out-of-plane bending stiffness of these walls is reduced by one-half to simulate cracked behavior in accordance with ASCE 43-05 (ASCE, 2005). For the walls located in the plane of symmetry, the modulus of elasticity and density are reduced by one-half to accurately represent mass and stiffness in the half model.

As shown in 10.4-4 and 10.4-5, the pump house enclosure and the electrical room for the Circulating Water Makeup Intake Structure are steel enclosures founded on grade slabs. The grade slabs are separated from the CBIS by providing an expansion joint, and are not included in the finite element model. The south end of the pump house enclosure is partially supported on the operating deck slab of the Circulating Water Makeup Intake Structure. The masses corresponding to the applicable dead loads and snow loads for the pump house enclosure are appropriately included in the finite element model.

The finite element model used for the seismic SSI analysis includes masses corresponding to 25 percent of floor design live load and 75 percent of roof design snow load, as applicable, and 50 pounds per square feet of miscellaneous dead load in addition to the self weight of the structure. The weights of equipment are included in the dynamic analysis.

The hydrodynamic effects of water contained in the CBIS are considered in accordance with ACI 350.3-06 (ACI, 2006). The impulsive and convective water masses due to horizontal earthquake excitation are calculated using the clear dimensions between the walls perpendicular to the direction of motion and for normal water level, corresponding to MSL, at El. 0.64 ft NGVD 29. The impulsive water masses are rigidly attached to the walls, and the convective water masses are connected to the walls using springs with appropriate stiffness. The entire water mass is lumped at the basemat nodes for earthquake ground motion in the vertical direction. The hydrodynamic loads are included for walls both in the Forebay and basement of the UHS Makeup Water Intake Structure.

The maximum sloshing heights in both directions for the UHS Makeup Water Intake Structure and the Forebay are approximately 0.82 ft (0.25 m) and 0.95 ft (0.29 m), respectively. The minimum available freeboard for the UHS Makeup Water Intake Structure and the minimum clearance for the Forebay are significantly higher than the maximum sloshing heights.

The earthquake excitation along the North-South and vertical directions cause symmetric loading on the structure, whereas the earthquake excitation along the East-West direction causes anti-symmetric loading on the structure. The seismic SSI analysis is performed by applying appropriate symmetric and anti-symmetric boundary conditions in the plane of symmetry of the half model shown in Figure 3.7-23, as indicated in Table 3.7-7.

3.7.2.3.3 Seismic Category II Structures

Insert for FSAR 3.7.2.3.3.

The Seismic Category II Circulating Water Makeup Intake Structure is analyzed along-with the Seismic Category I Forebay and Seismic Category I UHS Makeup Water Intake Structure, as described in Section 3.7.2.3.2. Other site-specific Seismic Category II structures, except the TI Structure, are designed using conventional codes and standards, but are also analyzed for Site SSE.

3.7.2.3.4 Conventional Seismic (CS) Structures

No departures or supplements.

3.7.2.4 Soil-Structure Interaction

This section describes the confirmatory soil-structure interaction (SSI) analyses for the Nuclear Island Common Basemat Structures, EPGB, and ESWB. In addition the SSI analysis of the CBIS, as well as the Seismic Category II Turbine Island (TI) Structure, are also described.

The complex frequency response analysis method is used for the SSI analyses, in accordance with the requirements of NUREG-0800 Section 3.7.2, Acceptance Criteria 1.A and 4 and Section 3.7.1, Acceptance Criteria 4.A.vii (NRC, 2007a). During the SSI analyses, the effects of foundation embedment (for ESWB and CBIS), soil layering, soil nonlinearity, ground water table, and variability of soil and rock properties on the seismic response of the structures are accounted for, as described in the following sections. In particular, Sections 3.7.2.4.1 through 3.7.2.4.6 provide the steps followed to perform the SSI analyses. Section 3.7.2.4.7 describes the computer codes used in the analyses.

Similarly, the SSI analysis of the TI Structure is performed using the complex frequency response analysis method described in detail in Sections 3.7.2.4.1 through 3.7.2.4.7.

3.7.2.4.1 Step 1 – SSE Strain Compatible Soil Properties

3.7.2.4.1.1 Nuclear Island Common Basemat Structures

For the Nuclear Island Common Basemat Structures, confirmatory SSI analyses are performed for the lower bound, best estimate and upper bound soil profiles established in Section 3.7.1.3.1 and shown in Table 3.7-2, Table 3.7-3 and Table 3.7-4. Soil properties used in the SSI

Insert for FSAR 3.7.2.3.3:

As described in Section 3.7.2.4.2.4, the SSI analysis of the Turbine Island Structure is performed using a finite element model.

The TI Structure is a site-specific structure and is classified as a nonsafety-related, Augmented Quality (NS-AQ) Seismic Category II structure according to COLA FSAR Table 3.2-1.

The sub-structure of TI Structure is comprised of a reinforced concrete basemat, below grade reinforced concrete walls and integral reinforced concrete pilasters supporting steel columns.

The super-structure of the TI Structure is primarily structural steel. However for the direction parallel to the Turbine Generator (TG), concrete shear walls are used below the operating deck.

For the direction parallel to the TG, above the operating deck, the Lateral Force Resisting System (LFRS) is a steel braced frame. For the direction perpendicular to the TG, the LFRS is steel braced frame below the operating deck with moment resisting frames above the operating deck.

Key floor levels of the TI Structure and their structural floor systems include:

- Grade Level: concrete slab supported by composite steel beams and either composite or non-composite girders.
- Mezzanine Level: for Turbine Building, structural steel framing supporting steel grating typically, with select areas of concrete slabs supported by structural steel beams and girders. For Switchgear Building, concrete slab supported by composite steel beams and either composite or non-composite girders.
- Battery Level: concrete slab supported by composite steel beams and either composite or non-composite girders.
- Operating Deck Level: concrete slab supported by composite or non-composite steel beams and girders depending on the location.
- Roof: for Turbine Building, structural steel beams and roof purlins supported by long-span structural steel roof trusses at main column lines. For Switchgear Building low roof, concrete slab supported by composite steel beams and either composite or non-composite girders. For Switchgear Building high roof, steel plate supported by beams and girders.

The Turbine Generator (TG) is supported by a reinforced concrete TG deck. The deck is supported by spring isolators. The spring isolators, grouped at primary column locations, are supported by structural steel box columns which are integral with the overall structural steel frame of the TI Structure.

The 3D finite element (FE) model of the TI Structure is developed in GTSTRUDL, Version 31, as shown in Figures 3.7-82 to 3.7-84. The FE model is used to generate the FE model for seismic SSI analysis using the Bechtel computer code SASSI2010, Version 1.1.

The finite element model in SASSI uses a thick shell element formulation that represents the in-plane and out-of-plane bending effects, as well as the in-plane and out-of-plane shear.

For the SSI analysis, the shear walls and end walls are considered to be cracked. In accordance with ASCE 43-05 (ASCE, 2005), the modulus of elasticity (E) and the shear modulus (G) are reduced to one-half of their values. Conservatively, the axial rigidity is also reduced by one-half. Considering the vertical load distribution system of the TI Structure, the reduction in axial rigidity of the shear walls and end walls will have insignificant effect on the global mass and stiffness distribution of the TI Structure.

The finite element model used for the seismic SSI analysis includes masses corresponding to 25 percent of the floor design live load and 75 percent of the roof design snow load. Major equipment (including Turbine Generator, Feedwater Tank, Moisture Separator Reheater, and Feedwater Heaters), is included explicitly in the dynamic analysis. An allowance is provided for piping, miscellaneous commodities, and minor equipment.

analysis are strain-compatible with the Site SSE, and account for the range of variation of shear-wave velocity, damping ratio, and P-wave velocity.

3.7.2.4.1.2 EPGB and ESWB

For the EPGB and ESWB, confirmatory SSI analyses are performed for the lower bound, best estimate and upper bound soil profiles established in Section 3.7.1.3.2. Table 3F-3, Table 3F-4, and Table 3F-5 show the properties for the top fifty layers of each soil profile (approximately 300 ft), while Figure 3F-29, Figure 3F-30 and Figure 3F-31, respectively, show the shear wave velocity, damping ratio and P-wave velocity for the top six hundred feet in this area. Soil properties used in the SSI analysis are strain-compatible with the Site SSE, and account for the range of variation of shear-wave velocity, damping ratio, and P-wave velocity.

3.7.2.4.1.4 Turbine Island Structure
As described in Section 3.7.1.3.4, the TI Structure shares the same supporting media with both the EPGB and ESWB. Soil properties used in the SSI analysis are strain compatible with the Site SSE, and account for the range of variation of shear wave velocity, damping ratio, and P-wave velocity.

3.7.2.4.1.3 Common Basemat Intake Structures

SSI analyses for the CBIS are performed for the lower bound, best estimate and upper bound soil profiles established in Section 3.7.1.3.3. Table 3F-6, Table 3F-7 and Table 3F-8 show the properties for the top fifty layers of each soil profile (approximately 380 ft), while Figure 3F-32, Figure 3F-33 and Figure 3F-34, respectively, show the shear wave velocity, damping ratio and P-wave velocity for the top six hundred feet in the intake area. Soil properties used in the SSI analysis are strain-compatible with the Site SSE, and account for the range of variation of shear-wave velocity, damping ratio, and P-wave velocity.

3.7.2.4.2 Step 2 – Development of Structural Model

3.7.2.4.2.1 Nuclear Island Common Basemat Structures

Confirmatory SSI analyses of the Nuclear Island Common Basemat Structures uses a surface founded stick model. 4 percent structural damping for reinforced concrete is used and 3 percent structural damping for pre-stressed concrete, NSSS components and vent stack is applied.

3.7.2.4.2.2 EPGB and ESWB

Confirmatory SSI analyses for the EPGB and ESWB use finite element models. 4% structural damping is used.

3.7.2.4.2.3 Common Basemat Intake Structures

Section 3.7.2.3.2 describes the development of the integrated finite element model of the CBIS in STAAD Pro, and translation of the model into SASSI. The thin plate element in SASSI is used to model all the structural panels.

The Common Basemat Intake Structures are reinforced concrete structures. A structural damping of 4 percent is used in the SSI analysis to obtain the ISRS, while 5 percent is used to obtain internal forces for the design of the CBIS using STAAD Pro.

Insert for FSAR 3.7.2.4.2.4.

3.7.2.4.3 Step 3 – Development of Soil Model

3.7.2.4.3.1 Nuclear Island Common Basemat Structures

SSI analyses are conducted for the three soil profiles discussed in Section 3.7.2.4.1.1, namely CCNPP Unit 3 strain-compatible BE, CCNPP Unit 3 strain-compatible LB and CCNPP Unit 3 strain-compatible UB. Each soil profile is discretized in a sufficient number of horizontal sub-layers, followed by a uniform half space beneath the lowest sub-layer.

Insert for FSAR 3.7.2.4.2.4:

3.7.2.4.2.4 Turbine Island Structure

Section 3.7.2.3.3 describes the development of the finite element model of the TI structure in GTSTRUDL. The model is translated into SASSI2010 Version 1.1 and the SSI analysis is performed to evaluate the seismic stability (overturning and sliding) of the TI.

With part of the TI Structure embedded in soil, excavated soil elements are modeled into the SASSI2010 model using eight-node brick elements. Since the direct method is used for the computation of the impedance matrix when performing SSI analysis, the nodes used to model the excavated soil elements are considered as interaction nodes.

To evaluate for overturning and sliding of the structure during a seismic event, translational spring elements are added to the building nodes below grade in contact with soil. These include the nodes for the basemat and the exterior walls adjacent to soil. The spring elements are modeled with very stiff spring properties. One end of the spring is attached to the structural shell element node, while the other end is attached to an interaction node in the soil.

The effect of ground water table on the seismic soil-structure-interaction (SSI) analysis of NI Common Basemat Structures is considered through modification of the P-Wave velocity profiles and by using the saturated weight for the soil below the ground water table.

3.7.2.4.3.2 EPGB and ESWB

The soil model is developed using the SSE strain-compatible lower bound, best estimate and upper bound soil profiles discussed in Section 3.7.2.4.1.2. Each soil profile is discretized in a sufficient number of horizontal sub-layers, followed by a uniform half space beneath the lowest sub-layer, which is located at a depth of 435 ft. The material soil or rock damping does not exceed 15 percent. P-wave damping is set to be equal to S-wave damping for all soil layers.

The effect of ground water table on the seismic soil-structure-interaction (SSI) analysis of the structure is considered through modification of the P-Wave velocity profiles as discussed in Section 3.7.1.3.2 and by using the saturated weight for the soil below the ground water table.

3.7.2.4.3.4 Turbine Island Structure
The soil model is developed using the SSE strain-compatible lower bound, best estimate and upper bound soil profiles , which are identical to the soil profiles for the EPGB and ESWB. Each soil profile is discretized in a sufficient number of horizontal sub-layers, followed by a uniform half space beneath the lowest sub-layer, which is located at a depth of 391 ft. The material soil or rock damping does not exceed 15 percent. P-wave damping is set to be equal to S-wave damping for all soil layers.
The effect of ground water table on the seismic soil-structure-interaction (SSI) analysis of the structure is considered through modification of the P-wave velocity profiles.

3.7.2.4.3.3 Common Basemat Intake Structures

The soil model is developed using the SSE strain-compatible lower bound, best estimate and upper bound soil profiles discussed in Section 3.7.2.4.1.3. Each soil profile is discretized in a number of horizontal sub-layers, based on shear propagation requirement, and a uniform half space is introduced beneath the lowest sub-layer, which is located at a depth of 350 ft. The material soil or rock damping does not exceed 15 percent. P-wave damping is set to be equal to S-wave damping for all soil layers.

The effect of ground water table on the seismic SSI analysis of the integrated CBIS is considered through modification of the P-Wave velocity profiles as discussed in Section 3.7.1.3.3, and by using the saturated weight for the soil below the ground water table.

3.7.2.4.4 Step 4 – Development of SSI Analysis Soil Model

3.7.2.4.4.1 Nuclear Island Common Basemat Structures

A surface founded stick model is used for the Nuclear Island Common Basemat Structures confirmatory SSI analyses. The analysis uses the following inputs:

- Site-specific soil profiles strain-compatible with the Site SSE are used, as described in Section 3.7.2.4.1.1.
- The free-field control input motion to the SSI analysis of the NI Common Basemat Structures is the Site SSE previously described in Section 3.7.1.1.2.1. The Site SSE is applied at NI foundation level, which is the horizon used for development of the NI FIRS (i.e., CCNPP Unit 3 GMRS described in Section 2.5.2.6). In particular, the surface outcrop motions (acceleration time histories) shown in Figure 3.7-10, Figure 3.7-11 and Figure 3.7-12 are used for the SSI analysis.
- Four percent structural damping is applied.

3.7.2.4.4.2 EPGB and ESWB

An SSI model and methodology of the EPGB and ESWB is used for the confirmatory SSI analyses. The analysis uses the following inputs:

- Site-specific soil profiles strain-compatible with the Site SSE are used, as described in Section 3.7.2.4.1.2.

- The control input motion for the SSI analysis of the EPGB and ESWB is the Site SSE described in Section 3.7.1.1.2.2. The control motion is applied at the foundation level (i.e., at the same horizon used for development of FIRS). In particular, for the EPGB, the surface outcrop motions (acceleration time histories) shown in Figure 3.7-10, Figure 3.7-11 and Figure 3.7-12 are used, while for the ESWB the within soil-column motions (acceleration time histories) shown in Figure 3.7-13, Figure 3.7-14 and Figure 3.7-15 are used.

Interaction forces are obtained at the basemat nodes at the soil-structure interface, and subsequently used in the stability analyses described in Section 3.7.2.14.2.

3.7.2.4.4.3 Common Basemat Intake Structures

The SSI model includes the CBIS, the surrounding layers of structural fill and the existing soil media as shown in Figure 3.7-24. Interaction forces are obtained at the basemat nodes at the soil-structure interface, and subsequently used in the stability analyses described in Section 3.7.2.14.2.

The control input motion for the SSI analysis of the CBIS is the within soil-column motion corresponding to the outcrop Site SSE for each soil profile, shown in Figures 3.7-16, 3.7-17 and 3.7-18 and described in Section 3.7.1.1.2.3. Consistent with the development of the within soil-column motion, the control motion is applied at the foundation level of the CBIS (i.e., at the same horizon used for development of FIRS for the CBIS).

Insert for FSAR 3.7.2.4.4.4.

3.7.2.4.5 Step 5 - Performing SSI Analysis

3.7.2.4.5.1 Nuclear Island Common Basemat Structures

Confirmatory SSI analyses for the Nuclear Island Common Basemat Structures are performed following the previously described methodology.

3.7.2.4.5.2 EPGB and ESWB

Confirmatory SSI analyses for the EPGB and ESWB are performed following the previously described methodology.

3.7.2.4.5.4 Turbine Island Structure
The SSI analysis of the model for the TI Structure is performed using Bechtel computer code SASSI2010, following the previously described methodology.

3.7.2.4.5.3 Common Basemat Intake Structures

The SSI analysis of the model for the CBIS is performed using RIZZO computer code SASSI. SSI analysis is performed for each direction of the Site SSE (i.e., X (N-S), Y (E-W), Z (Vertical)) and for each of the three soil profiles described in Section 3.7.2.4.1.3.

3.7.2.4.6 Step 6 - Extracting Seismic SSI Responses

3.7.2.4.6.1 Nuclear Island Common Basemat Structures

SSI analysis outputs are generated for each soil profile (i.e., LB, BE, and UB) and direction of the input motion. In particular in-structure response spectra for 5 percent damping are generated at the key locations as described in Section 3.7.2.5.1.

3.7.2.4.6.2 EPGB and ESWB

SSI analysis outputs are generated for each soil profile (i.e., LB, BE, and UB) and direction of the input motion. Accelerations, in-structure response spectra, and interaction forces at the soil-basemat interface are calculated.

Table 3.7-8 and Table 3.7-9 provide the combined average maximum nodal accelerations at various elevations of EPGB and ESWB, respectively. Comparison of the structural accelerations

Insert for FSAR 3.7.2.4.4.4:

3.7.2.4.4.4 Turbine Island Structures

The SSI model includes the TI Structure, the surrounding layers of structural fill, and the existing soil media. At each interaction node on the side walls and at the basemat, three orthogonal zero-length stiff spring elements link the structure to the soil. The spring forces due to dynamic loading are subsequently used in the stability analyses described in Section 3.7.2.8.

The control input motion for the SSI analysis of the TI Structure is the within soil column motion corresponding to the outcrop Site SSE for each soil profile described in Section 3.7.1.3.4. Consistent with the development of the soil column motion, the control motion is applied at the foundation level of the TI Structure (i.e., at the same horizon used for development of FIRS for the TI Structure).

provided in Table 3.7-8 and Table 3.7-9 with the corresponding structural accelerations reported in U.S. EPR FSAR Tables 3.7.2-27 and 3.7.2-28, respectively, show that the site-specific accelerations for EPGB and ESWB are bounded by the certified design.

Output response time histories of nodal interaction forces for each of the basemat nodes of the EPGB and ESWB are used to calculate response time histories of resultant sliding forces and overturning moments, which are used to evaluate the overall stability of each structure as described in Section 3.7.2.14.2.

In-structure response spectra are reported at selected locations of the EPGB and ESWB as detailed in Section 3.7.2.5.2.

3.7.2.4.6.3 Common Basemat Intake Structures

SSI analysis outputs are generated for each soil profile (i.e., LB, BE, and UB) and direction of the input motion. Accelerations, relative displacements, element forces, in-structure response spectra, resultant sliding force and total overturning moments are calculated.

Table 3.7-10 provides the combined maximum nodal accelerations at various elevations of UHS Makeup Water Intake Structure. These accelerations have been obtained using the methodology outlined in U.S. EPR FSAR Section 3.7.2.4.6.

Absolute peak element forces and moments (i.e., membrane and out-of-plane bending and shear resultants) are calculated for each soil profile and direction of the input motion. These forces and moments are used for the design of critical walls and slabs, as detailed in Appendix 3E.

For determination of seismic stability of the CBIS, the seismically induced normal and shear stresses at the base of the CBIS foundation are computed and compared with the restoring stresses from the self weight of the structure as described in Section 3.7.2.14.3.

In-structure response spectra (ISRS) are reported at selected locations of the CBIS as detailed in Section 3.7.2.5.3.

Editors Note: See Insert for FSAR 3.7.2.4.6.4.

3.7.2.4.7 Computer Codes

The confirmatory SSI analysis of the NI Common Basemat Structures is performed using AREVA computer code SASSI, Version 4.2; which has been verified and validated in accordance with the AREVA 10 CFR 50 Appendix B QA program.

~~Bechtel computer code SASSI2000, Version 3.1, is used to perform the seismic confirmatory SSI analysis of the EPGB and ESWB. This program is developed and maintained in accordance with Bechtel's engineering department and QA procedures. Validation manuals are maintained in the Bechtel Computer Services Library. The program is in compliance with the requirements of ASME NQA-1-1994.~~

RIZZO computer code SASSI, Version 1.3a, is used to perform the seismic confirmatory SSI analysis of the CBIS. This program is developed and maintained in accordance with RIZZO's engineering department and QA procedures. Validation manuals are maintained in the RIZZO Computer Services Library. The program is in compliance with the requirements of ASME NQA-1-1994.

[EPGB and ESWB analysis TBD.]

Bechtel computer code SASSI2010, Version 1.1, is used to perform the seismic SSI analysis of the TI structure. This program is developed and maintained in accordance with Bechtel's engineering department and QA procedures. Validation manuals are maintained in the Bechtel Computer Services Library. The program is in compliance with the requirements of ASME NQA-1-1994.

Insert for FSAR 3.7.2.4.6.4:

3.7.2.4.6.4 Turbine Island Structure

SSI analysis outputs are generated for each soil profile (i.e., LB, BE, and UB) and direction of the input motion. In particular, spring element forces, located at the soil-structure interface, are calculated.

Time-history responses are calculated by SASSI at a uniform time step of 0.005 seconds. For the spring elements at the bottom of the basemat and at the exterior walls adjacent to soil, the responses are spring forces in the three orthogonal translation directions. Two sets of results can be extracted from the SASSI analyses. The first set consists of only the maximum positive or negative forces for each spring. The second set consists of the actual resultant force for each spring at each time step, including the directional sign.

For the building stability evaluation of the Turbine Island Structure, only the maximum positive or negative forces for each spring are considered. An additional conservatism is to assume the spring forces are acting in the same direction (i.e. in-phase), using the absolute value of the output responses. Peak forces at each spring, in each response direction, due to seismic input in each of the 3 directions, and for each of the soil cases evaluated, are then combined. Within each soil case, since maximum responses are used in each response direction (X, Y, and Z), the responses due to X, Y, Z seismic inputs are combined using square-root-of-sum-of-squares (SRSS) method. The spring forces are similar to support reactions of the building and are used to calculate the total shear (sliding), total vertical uplift force, and overall overturning moment at the bottom of the basemat slab.

For stability evaluation using time-step methodology, resultant seismic inputs in each direction are combined algebraically at each time step for all nodes, while accounting for the directional sign (positive or negative). The sum of forces of all nodes, in each of the two horizontal directions, are then combined by the square-root-of-sum-of-squares (SRSS) to yield the total sliding demand force.

The determination of seismic stability of the TI Structure is presented in Section 3.7.2.8.

3.7.2.5.2 EPGB and ESWB

U.S. EPR FSAR Section 3.7.2.5 describes the development of floor response spectra for the EPGB and ESWB. The soil cases are described in U.S. EPR FSAR Table 3.7.1-6 and the ground design response spectra are shown in U.S. EPR FSAR Figures 3.7.1-33 and 3.7.1-34 for the EPGB and ESWB.

For site-specific confirmatory analysis, ISRS are generated for EPGB and ESWB at locations identified in U.S. EPR FSAR Section 3.7.2.5, using the guidelines described in U.S. EPR FSAR Section 3.7.2.5. The ISRS are however, calculated from 0.2 to 100 Hz, and correspond to the envelope of the ISRS for the site-specific strain-compatible BE, LB and UB soil profiles. For the purposes of confirmatory analyses, Figure 3.7-64 to Figure 3.7-72 show the comparison of 5 percent damped ISRS, which are representative of the response at all damping values, with the corresponding ISRS from U.S. EPR FSAR. The site-specific ISRS for these structures are enveloped by the corresponding design certification ISRS by a large margin, except for frequencies less than approximately 0.3 Hz. Reconciliation of the accelerations at these low frequencies is discussed in Section 2.5.2.6.

3.7.2.5.3 Common Basemat Intake Structures

ISRS at the location of safety-related equipment within the UHS Makeup Water Intake Structure are generated using the SSI model described in Section 3.7.2.4. The ISRS are calculated from 0.1 to 50 Hz, which meets the guidelines provided in RG 1.122, Revision 1 (NRC, 1978). For the UHS Makeup Water Intake Structure, the ISRS are calculated at 0.5 percent, 2 percent, 3 percent, 4 percent, 5 percent, 7 percent and 10 percent damping. The ISRS are enveloped for the site-specific strain-compatible BE, LB and UB soil profiles.

For the UHS Makeup Water Intake Structure, the ISRS are developed at the location of safety-related makeup pumps and facilities, as shown in Figure 3.7-73 through Figure 3.7-78 and at the location of safety-related electrical equipment supported at EL +26.5 ft in the CBIS, and are shown in Figure 3.7-79 through Figure 3.7-81. ISRS will be generated at the support locations of additional safety-related equipment, as required.

3.7.2.6 Three Components of Earthquake Motion

As indicated in Section 3.7.2.4, the SSI analysis of the site-specific Seismic Category I structures is performed using the integrated finite element model, with the input ground motion applied separately in the three directions. The ISRS in the UHS Makeup Water Intake Structure are determined by using the Square Root of Sum of Squares (SRSS) of the calculated response spectra in a given direction, due to earthquake motion in the three directions.

The maximum member forces and moments due to the three earthquake motion components are combined using the Square Root of the Sum of the Squares (SRSS) combination rule to obtain the maximum total member forces and moments. The SRSS method rule used is consistent with the requirements of RG 1.92, Revision 2 (NRC, 2006).

3.7.2.7 Combination of Modal Responses

No departures or supplements.}

3.7.2.8 Interaction of Non-Seismic Category I Structures with Seismic Category I Structures

The U.S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.7.2.8:

A COL applicant that references the U.S. EPR design certification will provide the site-specific separation distances for the Access Building and Turbine Building.

The COL Item is addressed as follows:

The conceptual design information in U.S. EPR FSAR, Tier 2, Figure 3B-1 provides the separation gaps between the AB and SBs 3 and 4 and between the TB and the NI Common Basemat Structures. This information is incorporated by reference.

The U. S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.7.2.8 - Access Building:

A COL applicant that references the U.S. EPR design certification will demonstrate that the response of the Access Building to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions.

[[The Access Building is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Category I structure with the exception of sliding and overturning criteria. Because the Access Building does not have a safety function, it may slide or uplift provided that the gap between the Access Building and any Category I structure is adequate to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the Access Building and adjacent Category I structures. The separation gaps between the Access Building and SBs 3 and 4 are 0.98 ft and 1.31 ft, respectively (U.S. EPR FSAR, Tier 2, Figure 3B-1).]]

For COL applicants that incorporate the conceptual design for the Access Building presented in the U.S. EPR FSAR (i.e ., [[the Access Building is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Category I structure with the exception of sliding and overturning criteria]]), this COL item is addressed by demonstrating that the gap between the Access Building and adjacent Category I structures is sufficient to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the Access Building and adjacent Category I structures.

This COL Item is addressed as follows:

{The Access Building is classified as Seismic Category II structure and will be designed to satisfy SRP 3.7.2 Acceptance Criterion 8.C.}

The U. S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.7.2.8 - Turbine Building:

A COL applicant that references the U.S. EPR design certification will demonstrate that the response of the TB (including Switchgear Building on the common basemat) to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions.

Enclosure 2
UN#13-086

TI Structure

[[The ~~TB~~ is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Category I structure ~~with the exception of sliding and overturning criteria. Because the TB does not have a safety function, it may slide or uplift provided that the gap between the TB and any Category I structure is adequate to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the TB and adjacent Category I structures.~~ The separation between the ~~TB~~ and NI Common Basemat Structures is approximately 30 ft (U.S. EPR FSAR, Tier 2, Figure 3B-1).]]

TI Structure

~~For COL applicants that incorporate the conceptual design for the TB presented in the U.S. EPR FSAR (i.e., [[the TB is analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Category I structure with the exception of sliding and overturning criteria]]), this COL item is addressed by demonstrating that the gap between the TB and adjacent Category I structures is sufficient to prevent interaction. The effects of sliding, overturning, and any other calculated building displacements (e.g., building deflections, settlement) must be considered when demonstrating the gap adequacy between the TB and adjacent Category I structures.~~

This COL Item is addressed as follows:

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{The Turbine Building and Switchgear Building (also referred to as the Turbine Island (TI) structure) are classified as Seismic Category II structures. These structures were analyzed and designed to the same requirements as other Seismic Category I structures for site-specific SSE loads. This design methodology meets the NUREG 0800 SRP 3.7.2 Acceptance Criterion 8.C.}

Insert for FSAR 3.7.2.8.

The U.S. EPR FSAR includes the following COL Item and conceptual design information in Section 3.7.2.8 - Fire Protection Storage Tanks and Buildings:

> A COL applicant that references the U.S. EPR design certification will provide the seismic design basis for the sources of fire protection water supply for safe plant shutdown in the event of a SSE.

[[The Fire Protection Storage Tanks and Buildings are classified as Conventional Seismic Structures.]] RG 1.189 requires that a water supply be provided for manual firefighting in areas containing equipment for safe plant shutdown in the event of a SSE. [[The fire protection storage tanks and building are designed to provide system pressure integrity under SSE loading conditions. Seismic load combinations are developed in accordance with the requirements of ASCE 43-05 using a limiting acceptance condition for the structure characterized as essentially elastic behavior with no damage (i.e., Limit State D) as specified in the Standard.]]

The COL Item is addressed as follows:

{The U.S EPR FSAR Section 3.7.2.8 states that the Fire Protection Storage Tanks and Buildings are classified as Conventional Seismic Structures and that RG 1.189 (NRC, 2007) requires that a water supply be provided for manual firefighting in areas containing equipment for safe plant shutdown in the event of a SSE. The U.S. EPR FSAR Section 3.7.2.8 also states the fire protection

Insert for FSAR 3.7.2.8:

Table 3.7-10a summarizes the results for the stability evaluation of the Turbine Island Structure. For sliding assessment, the maximum required coefficient of friction against sliding of the structure is shown for each soil case. Shear resistance against sliding is provided by the weight of the structure minus the net uplift force from vertical seismic, multiplied by the coefficient of friction factor.

In the Upper Bound case, following the time step methodology described in Section 3.7.2.4.6.4, the actual sliding effects are calculated at each time step of 0.005 seconds. As a result of this evaluation, the required coefficient of friction for the Upper Bound soil case is 0.19 (see Table 3.7-10a and Figure 3.7-85). The calculated factor of safety (FOS) is therefore 2.68 for the Upper Bound case.

Table 3.7-10a indicates that the maximum required coefficients of friction for the Lower Bound and Best Estimate soil cases are 0.33 and 0.45, respectively. These coefficients of friction are calculated using maximum absolute demand forces as described in Section 3.7.2.4.6.4. The resulting FOS are 1.16 and 1.58 for the Lower Bound and Best Estimate soil cases, respectively. All factors of safety are larger than the required FOS of 1.1 for SSE per NUREG 0800 Standard Review Plan (SRP) 3.8.5. However it is important to note that in the two latter cases, conservatism is included in the analysis, and it is expected that if, similar to the Upper Bound case, time-step methodology were used, the calculated factors of safety would be larger than reported.

The table also summarizes the results for the uplift and overturning evaluations of the TI Structure. Demands, capacities, and factors of safety against uplifting and overturning of the structure are shown. For the structural overturning stability assessment, overturning moment effects about the four sides of the basemat slab are considered individually. Capacity against uplift is provided by the static weight of the structure. The resulting minimum FOS for uplift is 1.98 in the Upper Bound soil case, which is larger than the required FOS of 1.1 for SSE per NUREG 0800 SRP 3.8.5.

Capacities against overturning are the resisting moments provided by the structure weight, separately bending about the four sides of the basemat. The summary table indicates the minimum computed Factor of Safety (FOS) for overturning in all three soil cases is 1.82, which is more than the required 1.1 for SSE per SRP 3.8.5. Since the computed factor of safety for overturning is more than required, further evaluation using the time-step methodology is not needed.

In addition to determining the structural stability of the TI, an evaluation is performed to establish whether separation occurs between the concrete basemat and the soil. The evaluation consists of identifying the locations of the various springs where a net uplift force occurs. This is done by conservatively assuming the maximum seismic vertical force for all springs are applied in the upward direction at the same time. For each spring, a net uplift occurs when the seismic vertical force exceeds the static weight tributary for that spring. The results are shown in Figure 3.7-86 for the most critical Upper Bound soil case. The figure shows the locations of the springs at the basemat, along with the locations of the springs where net uplift force results. As expected, uplift occurs for the springs along the perimeter of the basemat. This is reasonable since the seismic overturning effects will be maximized at the edge of the slab. However, this localized separation between the concrete basemat and the soil

is not expected to be critical, due to the fact that the springs in the areas adjacent to the perimeter are experiencing net compressive forces. The local uplift will only occur for a short time interval, and then weight from the adjacent concrete area will force the slab back down. Figure 3.7-86 also shows that net uplift occurs within some localized interior regions of the basemat. Similar to the reasoning for the perimeter uplift, it is expected that weight from the adjoining areas will offset these uplift effects. It should be noted that the separation evaluation is performed using the maximum seismic uplift spring forces occurring at the same time. If time-step methodology is used, then the number of springs with net uplift is expected to be reduced, along with the possibility that these springs will be scattered randomly throughout the slab, instead of concentrated within one area.

storage tanks and building are designed to provide system pressure integrity under SSE loading conditions.

Refer to Section 3.2.1 and U.S. EPR FSAR Section 3.2.1 for further discussion of seismic classifications. In addition, Section 3.2.1 categorizes Fire Protection SSC into two categories:

1. SSC that must remain functional during and after an SSE; and
2. SSC that must remain intact after an SSE without deleterious interaction with any Seismic Category I SSC.

U.S. EPR FSAR Section 3.7.2.8 addresses the interaction of the following Non-Seismic Category I structures with Seismic Category I structures:

- Nuclear Auxiliary Building
- Access Building
- Turbine Building
- Radioactive Waste Processing Building

{The following CCNPP Unit 3 Non-Seismic Category I structures identified in Table 3.2-1 could also potentially interact with Seismic Category I SSC:

- Buried and above ground Seismic Category II Fire Protection SSC, including Fire Water Storage Tanks and Fire Protection Building.
- Seismic Category II Turbine Building and Switchgear Building
- Seismic Category II Access Building
- Conventional Seismic Grid Systems Control Building
- Seismic Category II Circulating Water Makeup Intake Structure
- Conventional Seismic Sheet Pile Wall.
- Existing Baffle Wall.

)

TI Structure
(i.e., combined

The buried Fire Protection SSC identified in Table 3.2-1 are seismically analyzed using the design response spectra identified in Section 3.7.1.1.1.4. These piping mains will be designed according to ASCE 4-98, 1983 ASCE Report "Seismic Response of Buried Pipes and Structural Components," and the Areva Topical Report ANP 10264, "U.S. EPR Piping Analysis and Pipe Support Design Topical Report."

The above ground Seismic Category II and Fire Protection SSC, including Fire Water Storage Tanks and Fire Protection Building, identified in Table 3.2-1 are seismically analyzed utilizing the appropriate design response spectra. Seismic load combinations are developed in accordance with the requirements of ASCE 43-05 (ASCE, 2005) using a limiting acceptance condition for the structure characterized as essentially elastic behavior with no damage (i.e., Limit State D) as specified in the Standard. The analysis of the above ground fire protection SSC will confirm they remain functional during and following an SSE in accordance with NRC

Regulatory Guide 1.189 (NRC, 2007). The analysis of the above ground Seismic Category II fire protection SSCs will confirm they maintain a pressure boundary after an SSE event.

Table 3.7-11 provides the criteria used to prevent seismic interaction of ~~Turbine Building~~ the TI Structure, ~~Switchgear Building~~, Access Building, Circulating Water Makeup Intake Structure and Grid Systems Control Building with other Seismic Category I structures, systems and components (SSCs).

The Seismic Category II Turbine Building (TB), Switchgear Building (SWGB) and Access Building (AB) are located in the vicinity of the Nuclear Island Common Basemat Structures. These buildings are analyzed and designed to prevent their failure under site-specific SSE loading conditions and to maintain margin of safety equivalent to that of Seismic Category I structures. The structural steel components of these structures are designed using ANSI/AISC N690 (ANSI/AISC, 2004). The reinforced concrete components of these structures are designed using ACI 349 (ACI, 2001). Therefore, the design methodology for these structures meets NUREG-0800 Section 3.7.2, Acceptance Criterion 8.C (NRC, 2007a). During detailed design, the elastic displacements of the TB, the SWGB and the AB will be computed using classical finite element analysis methods. The elastic displacements will be combined with those of the nearest Seismic Category I structures. It will be confirmed that the combined elastic displacements are less than the provided separation distances.

The analysis and design of the Seismic Category II ~~Turbine Building, Switchgear Building,~~ the TI Structure and Access Building for loads and load combinations including both seismic and non-seismic loads, are performed to the same requirements as Seismic Category I structures. The analysis of these structures is performed using three-dimensional finite element models, and the design of reinforced concrete and structural steel components is performed using ACI 349 (ACI, 2001) and ANSI/AISC N690 (ANSI/AISC, 2004) respectively. Therefore, these Seismic Category II structures have a margin of safety equivalent to that of Seismic Category I structures for applicable loads and load combinations. The structural acceptance criteria for these structures is consistent with those for Seismic Category I structures.

Turbine Island (TI) Structure (Turbine Building and Switchgear Building)

1. Building Layout/Details

~~The Turbine Building and Switchgear Building together comprise the Turbine Island (TI) structure. The TI structure is a site-specific structure and is classified as a Non Safety-Related Augmented Quality (NS-AQ) Seismic Category II structure according to COLA FSAR Table 3.2-1.~~

~~The TI sub-structure is comprised of a reinforced concrete basemat and below grade reinforced concrete walls. Integral reinforced concrete pilasters are situated beneath steel columns along the perimeter walls. The TI super-structure is primarily a structural steel, braced frame in both directions below the operating deck and parallel to the Turbine Generator (TG) above the operating deck. Structural steel moment frames are used perpendicular to the TG above the operating deck. A three-hour rated fire barrier separates the Turbine Building from the adjacent Switchgear Building.~~

~~Key floor levels of the TI structure and their structural floor systems include:~~

- ~~Turbine Building and Switchgear Building share a common Reinforced Concrete Basemat~~

- ~~Basement Level: Top of the reinforced concrete TI structure basemat~~
- ~~Grade Level: Composite slab supported by composite beams and either composite or non-composite girders~~
- ~~Mezzanine Level: Structural steel framing supporting steel grating typically, with select areas of composite slabs, composite beams and girders~~
- ~~Operating Deck Level: Reinforced concrete slab supported by composite beams and either composite or non-composite girders.~~
- ~~Roof: Structural steel beams and roof purlins supported by long-span structural steel roof trusses at main column lines~~

~~The Turbine Generator (TG) is located at Operating Deck Level. The reinforced concrete TG deck is supported by spring isolators. The spring isolators, grouped at primary column locations, are supported by either structural steel box columns or composite columns, which are integral with the overall structural steel frame of the TI structure.~~

The combined TI Structure is described in Section 3.7.2.3.3.

2. Acceptable Codes and Standards

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The TI structure is analyzed and designed to the same requirements as the site-specific Seismic Category I structures. The applicable design codes and standards for the reinforced concrete and structural steel components of the TI structure are listed in COLA FSAR Tables 3.2-1 and 3.7-11. The reinforced concrete components of the TI structure are designed in accordance with ACI 349 and the structural steel components of the TI structure designed in accordance with ANSI/AISC N690.

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3. Loads and Load Combinations

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The design loads and load combinations for the TI structure are consistent with those used for the design of site-specific Seismic Category I structures.

a. Dead Loads

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Dead loads for the TI structure are in accordance with Section 3.8.4.3.1, which incorporates the U.S. EPR FSAR Section 3.8.4.3.1 by reference.

b. Live Loads

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Live loads for the TI structure are in accordance with Section 3.8.4.3.1. The live loads include the loads due to rain, snow and ice and are based on the site-specific conditions. The live loads of the TI structure further include loads consistent with the equipment layout within the structure.

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c. Seismic Loads

Seismic loads are calculated using the CCNPP Unit 3 Site SSE Spectrum, which is depicted in COLA FSAR Figure 3.7-1.

d. Wind Loads

The TI structure is a site-specific structure and is designed for site-specific wind loads. The site-specific wind parameters (i.e. basic wind speed, 100-year recurrence interval wind speed) and applicable codes and guidelines ~~(e.g., importance factor of 1.15)~~ for the determination of site-specific wind loads were included in Section 3.3.1.

e. Tornado Loads

S

Tornado loads for the analysis and design of TI structure are in accordance with COLA FSAR Section 3.3.2. Design basis tornado characteristics and tornado missile parameters are in accordance with Tornado Region I of NRC RG 1.76, Revision 1. Tornado wind loads will be converted to wind pressure loads according to SEI/ASCE 7-05 guidelines. Tornado wind pressures are mitigated through the use of pressure relief siding panels.

f. Load Combinations

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Load combinations for the analysis and design of the TI structure are consistent with those used for the analysis and design of Seismic Category I structures, as described in Section 3.8.4.3.2, which incorporates the U.S. EPR FSAR Section 3.8.4.3.2 by reference.

4. Analysis Procedures

S

Finite element methods are utilized to analyze the TI structure for applicable loads and load combinations. For seismic loads, the TI structure is designed to maintain a margin of safety equivalent to that of Seismic Category I structures. The margin of safety equivalent to that of Seismic Category I structures is achieved by analyzing and designing the TI structure to the same requirements as a Seismic Category I structure.

5. Materials

Concrete and reinforcing steel materials for the TI structure conform to those for site-specific Seismic Category I structures which are discussed in COLA FSAR Section 3E.4.2. The concrete used for the TI structure has a minimum design compressive strength of ~~5000~~ psi (with ASTM A615, Grade 60 or 75 reinforcing steel). The structural steel W-shape members are ASTM A992, and the structural steel channels, angles, and plates conform to ASTM A36 (except angles of cased seats for modular composite floor panels which conform to ASTM A572 Grade 50).

4000

6. Acceptance Criteria

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Since the TI structure is designed to the same requirements as the Seismic Category I structures, the structural acceptance criteria for the TI structure is identical to those for Seismic Category I structures, which are outlined in COLA FSAR Section 3.8.4.5, which incorporates the U.S. EPR FSAR Section 3.8.4.5 by reference.

The TI structure is designed to remain elastic under an SSE. Therefore, in regards to seismic interaction considerations, the design methodology for the TI structure meets NUREG-0800, Standard Review Plan 3.7.2, Acceptance Criterion 8.C. The elastic displacements of the TI structure are computed using finite element analysis methods. Upon closure of COLA Part 10 Appendix B ITAAC, Revision 8, in Tables 2.4-10 and 2.4-11, the finite element analyses report will confirm that the elastic displacements of

time by computing the factors of safety as the ratio of the restoring stresses of the CBIS to the corresponding seismically induced stresses.

The factors of safety evaluated for the seismic stability are compared with the minimum required factors of safety specified in U.S. EPR FSAR Table 3.8-11. According to this reference, the minimum required factors of safety for sliding and overturning associated with Safe Shutdown Earthquake (E', Seismic Category I foundations) loading combination is 1.1. As a result the CBIS are evaluated to be safe against sliding and overturning due to seismic loads. Results of dynamic stability are reported in Appendix 3E.

The structure and soil damping used in the SSI analysis of site specific Turbine Island Structure is described in Section 3.7.1.2.

3.7.2.15 Analysis Procedure for Damping

The structure and soil damping used in SSI analyses of site-specific Seismic Category I structures are described in Sections 3.7.2.4.2.3 and 3.7.2.4.3.3.

3.7.2.16 References

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

ACI, 2001. 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.

ANSI/AISC, 2004. Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities, 1994 including Supplement 2, ANSI/AISC N690, American National Standards Institute, 2004.

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

ASCE, 2005. Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, ASCE 43-05, American Society of Civil Engineers, January 2005.

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

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

NRC, 2006. Combining Modal Responses and Spatial Components in Seismic Response Analysis, Regulatory Guide 1.92 Revision 2, U.S. Nuclear Regulatory Commission, July 2006.

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

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

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

Table 3.7-10— {Worst Case Accelerations in Common Basemat Intake Structures}

UHS Makeup Water Intake Structure			
Floor Elevation	**X (N-S) Direction**	**Y (E-W) Direction**	**Z (Vert) Direction**
22.5	0.225g	0.147g	0.233g
11.5	0.315g	0.199g	0.238g
26.5	0.342g	0.236g	0.240g
Forebay			
Floor Elevation	**X (N-S) Direction**	**Y (E-W) Direction**	**Z (Vert) Direction**
-22.5	0.227g	0.153g	0.215g

Note:
Elevations and plant coordinate system refer to U.S EPR FSAR.

Editors Note: See Insert for New Table 3.7-10a.

Insert for new Table 3.7-10a:

Table 3.7-10a— {Summary of Results - Building Stability Evaluation of the TI Structure}

SOIL CASE	COMPONENT	DEMAND (1) (kips) or (kip-ft)	REQUIRED (2) COEFFICIENT OF FRICTION, μ_{req}	CAPACITY (3)	FACTOR OF SAFETY = CAPACITY ÷ DEMAND
Lower Bound	Maximum Shear at Bottom of Basemat	89,999	0.33	0.52	1.58
	Vertical (Uplift) Force at Bottom of Basemat	99,204	—	311,208	3.14
	Overturning Moment Bending About Top Edge of Basemat	14,341,387	—	46,326,023	3.23
	Overturning Moment Bending About Bottom Edge of Basemat	13,635,147	—	38,322,600	2.81
	Overturning Moment Bending About Left Edge of Basemat	22,383,277	—	57,587,531	2.57
	Overturning Moment Bending About Right Edge of Basemat	16,825,248	—	62,850,032	3.74
Best Estimate	Maximum Shear at Bottom of Basemat	117,427	0.45	0.52	1.16
	Vertical (Uplift) Force at Bottom of Basemat	127,545	—	311,208	2.44
	Overturning Moment Bending About Top Edge of Basemat	19,084,957	—	46,326,023	2.43
	Overturning Moment Bending About Bottom Edge of Basemat	16,916,867	—	38,322,600	2.27
	Overturning Moment Bending About Left Edge of Basemat	27,059,275	—	57,587,531	2.13
	Overturning Moment Bending About Right Edge of Basemat	23,380,397	—	62,850,032	2.69
Upper Bound	Maximum Shear at Bottom of Basemat	57180 (4)	0.19 (5)	0.52	2.74
	Vertical (Uplift) Force at Bottom of Basemat	157,475	—	311,208	1.98
	Overturning Moment Bending About Top Edge of Basemat	23,951,116	—	46,326,023	1.93
	Overturning Moment Bending About Bottom Edge of Basemat	20,507,618	—	38,322,600	1.87
	Overturning Moment Bending About Left Edge of Basemat	31,586,890	—	57,587,531	1.82
	Overturning Moment Bending About Right Edge of Basemat	30,707,819	—	62,850,032	2.05

(1) Unless noted otherwise, the demands shown are based on the summation of the maximum resultant force for each spring, even though the maximum for different springs will occur at different time steps. The spring force directional sign (positive or negative) is conservatively neglected, assuming that all spring forces are acting in the same direction.

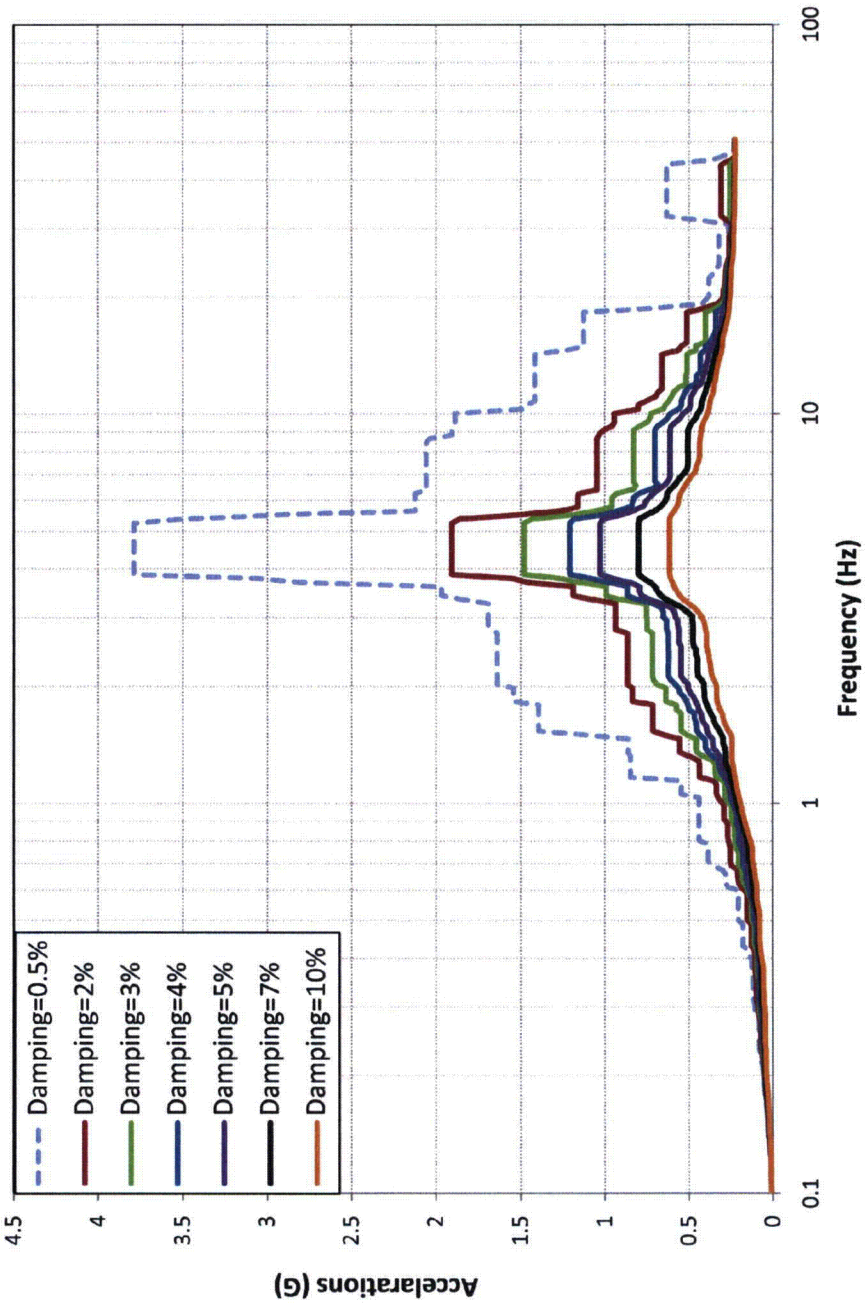
(2) Required coefficient of friction, μ_{req} = Shear demand ÷ [Structural Weight - (0.4 * Uplift Demand)]

(3) The sliding capacity is the available coefficient of friction at the interface below the TI basemat. The overturning moment capacities are the resisting moments provided by the structural weight bending about the four edges of the basemat. The passive soil resistances provided by the side soils against sliding and overturning are conservatively neglected.

(4) Value shown is the calculated shear demand for the evaluation case using time-step method, where calculation is performed at each time-step (0.005 seconds).

(5) Value shown is the calculated required coefficient of friction for the evaluation case using time-step method, where calculation is performed at each time-step (0.005 seconds).

Figure 3.7-81— {ISRS for Makeup Water Intake Structure at Elev. 26.5 ft (8.08 m), Vertical Direction. Elevations and plant coordinate system refer to CCNPP Unit 3}



Editors Note: See Insert for new Figures 3.7-82 to 3.7-86

Insert for new Figures 3.7-82 to 3.7-86: -

Figure 3.7-82 — {View of Typical Bay Perpendicular to the Turbine Generator (GT Strudl Finite Element Model)}

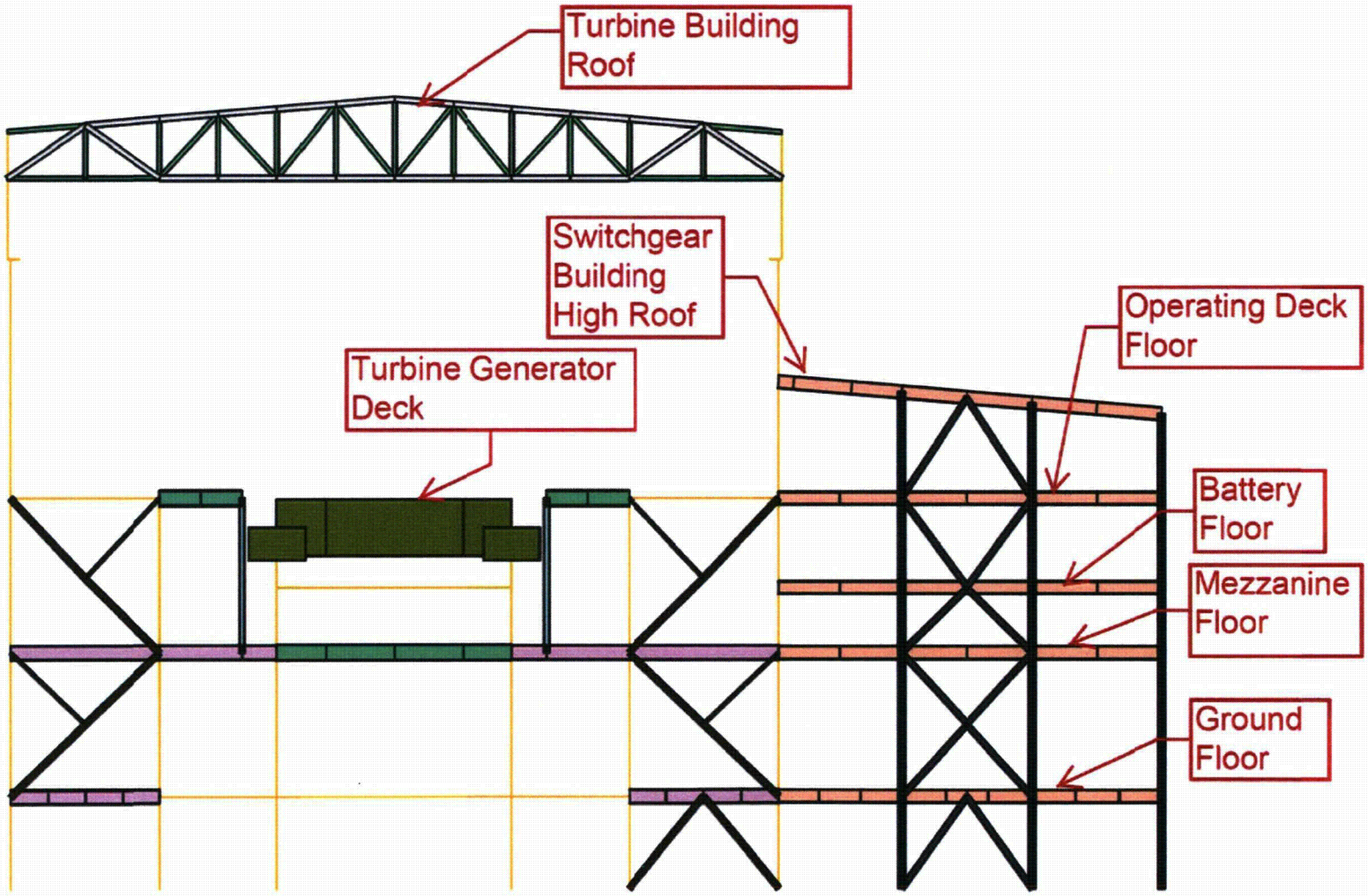


Figure 3.7-83 — {Cut - Isometric View of the Turbine Island Structure with shear walls (GT Strudl Finite Element Model)}

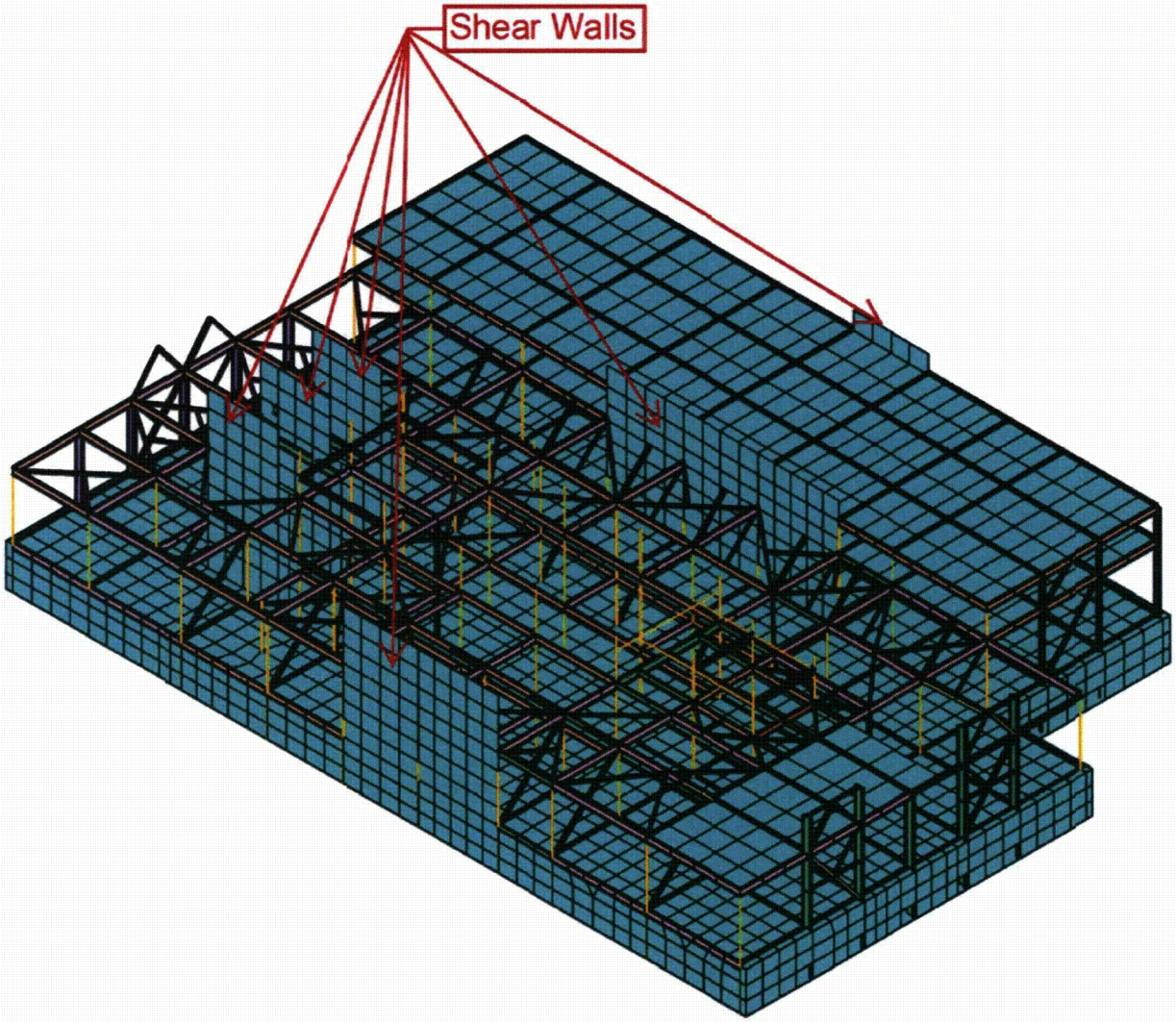


Figure 3.7-84 — {Isometric View of the Turbine Island Structure (GT Strudl Finite Element Model)}

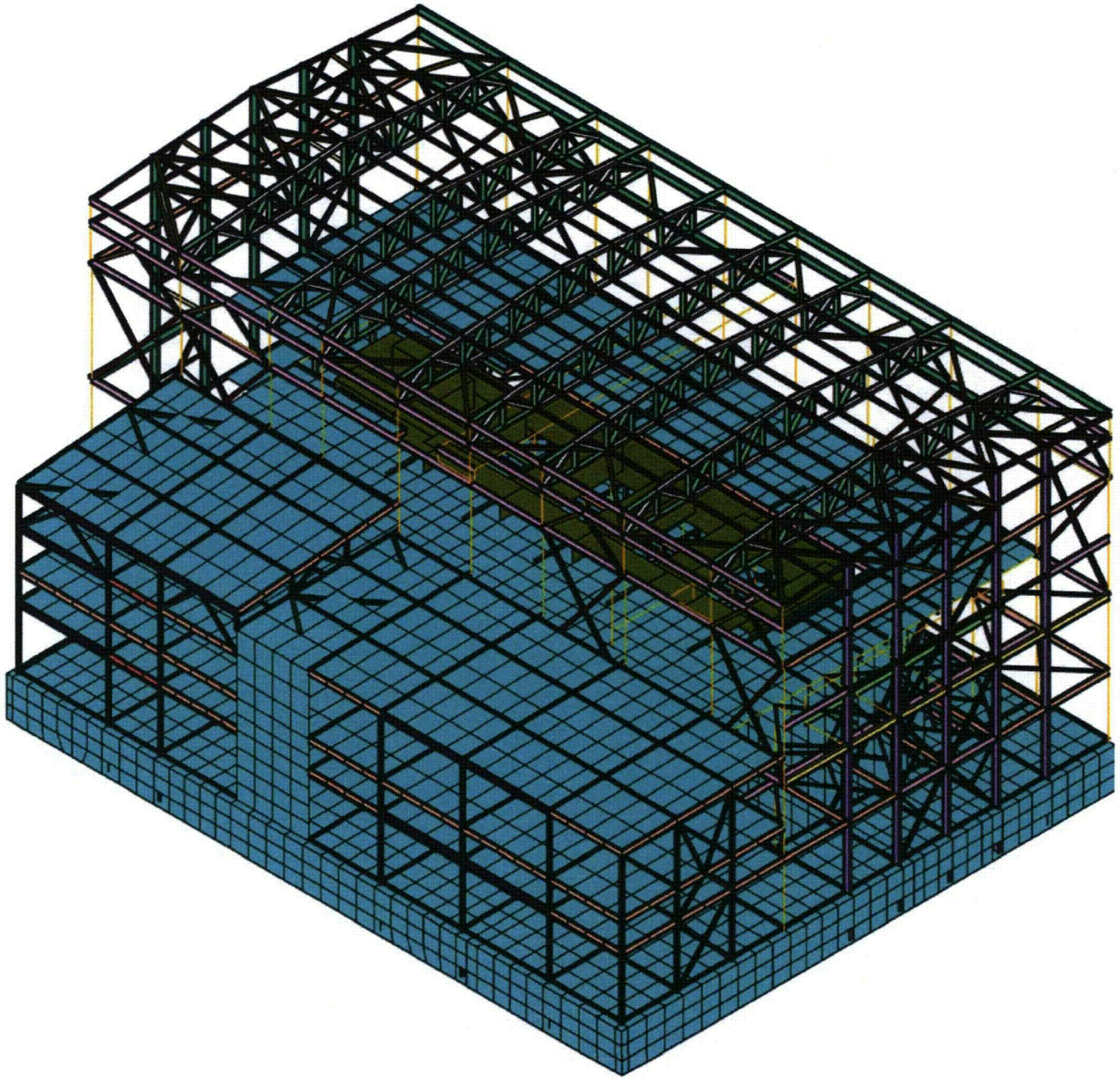


Figure 3.7-85 — {Turbine Island Upper Bound Soil Case - Required Coefficient of Friction for Sliding}

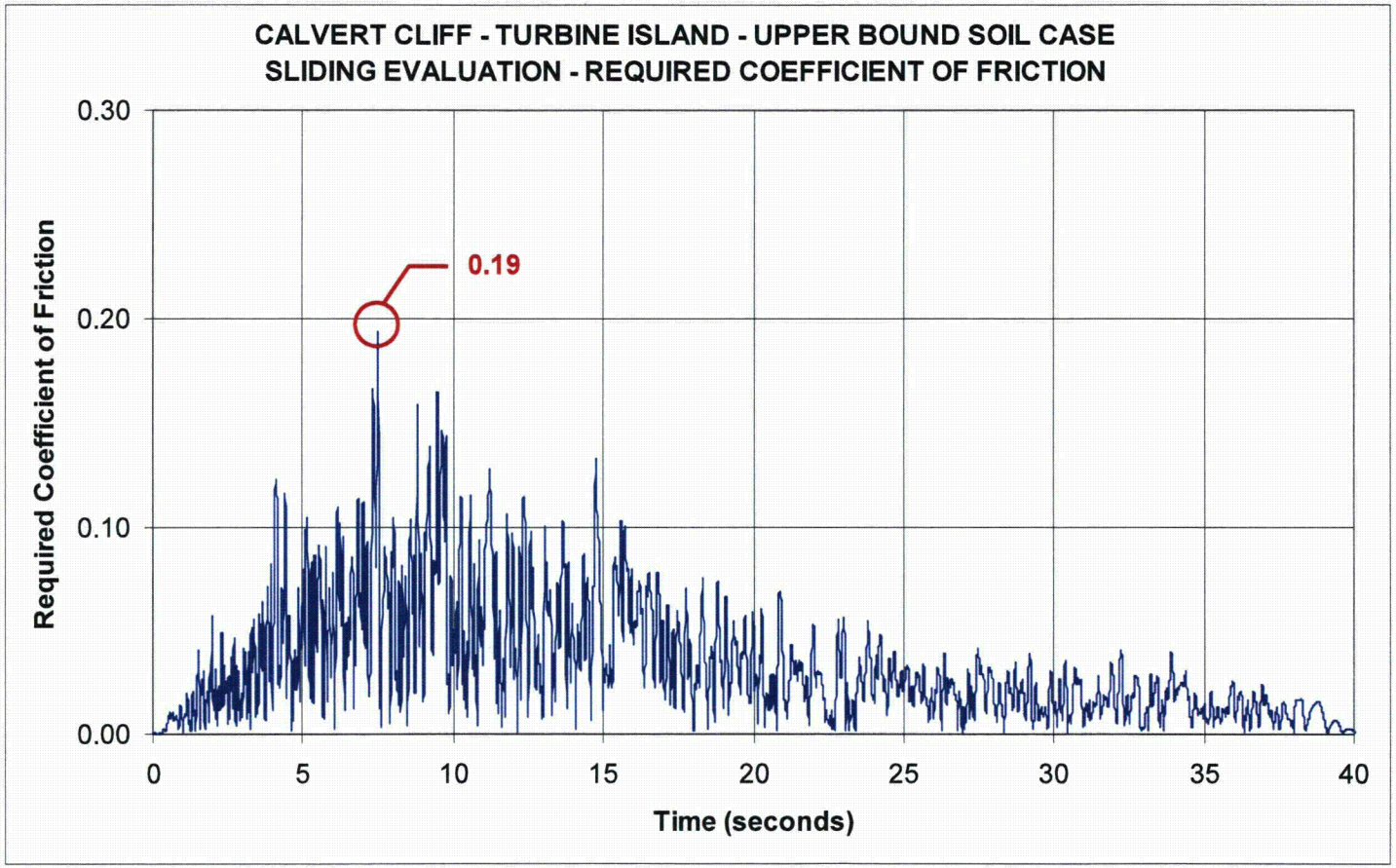


Figure 3.7-86 — {Turbine Island Upper Bound Soil Case - Springs With Net Tension at Basemat}

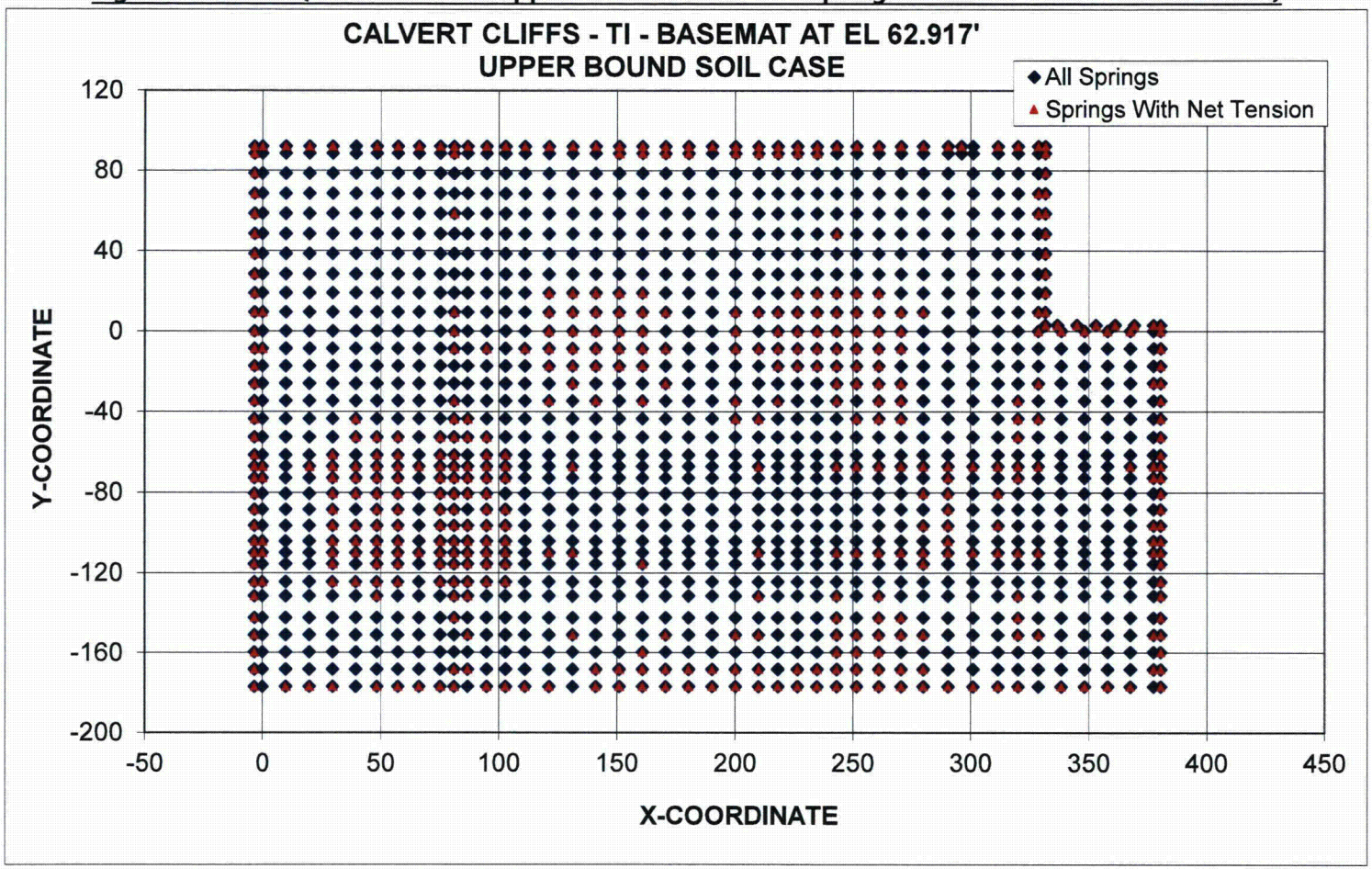


Table 3.8-4— {Factors of Safety for NI Common Basemat Structure, EPGB, and ESWB under SSE Loading}

Turbine Island Structure,

Building	Sliding	Overturning
NI Common Basemat Structure	1.88	4.50
EPGB	1.77	3.17
ESWB	3.19	4.0
Turbine Island Structure	1.16	1.82

Table 2.4-10— {Turbine Building Inspections, Tests, Analyses, and Acceptance Criteria}

	Commitment Wording	Inspection, Test, or Analysis	Acceptance Criteria
1	a. The Turbine Building is located in a radial position with respect to the Reactor Building, but is independent from the Nuclear Island. b. The Turbine Building is oriented to minimize the effects of any potential turbine generated missiles.	a. An inspection of the as-built structure will be performed. b. An analysis of the as-built structure's location and orientation will be performed.	a. The as-built Turbine Building location is in a radial position with respect to the as-built Reactor Building, and is independent from the as-built Nuclear Island. b. The as-built Turbine Building's location and orientation are consistent with the assumptions utilized in the analysis of the potential turbine missiles.
2	The Turbine Building does not impact the ability of any safety-related structure, system, or component to perform its safety function under design basis loads, as specified below. ♦ Normal plant operation (including dead loads, live loads, lateral earth pressure loads, equipment loads, hydrostatic, hydrodynamic, and temperature loads). ♦ Internal events (including internal flood loads, accident pressure loads, accident thermal loads, accident pipe reactions, and pipe break loads, including reaction loads, jet impingement loads, cubicle pressurization loads, and missile impact loads). ♦ External events (including wind, rain, snow, flood, tornado, tornado-generated missiles and earthquake).	An inspection and analysis will be performed to verify the as-built Turbine Building will withstand design basis loads.	~~A report concludes that the Turbine Building will not impact the ability of any safety-related structure, system or component to perform its safety function under design basis loads, as specified below. The report also concludes that the design of the Turbine Building is to the same requirements as a Seismic Category I structure~~ Insert for ITAAC Table 2.4-10.
3	The Turbine Building houses the components of the steam condensate main feedwater cycle, including the turbine-generator.	An inspection of the as-built structure will be performed.	The as-built Turbine Building houses the components of the steam condensate main feedwater cycle, including the turbine-generator, in accordance with the design.
4	For the Turbine Building's below grade concrete foundation and walls exposed to ground water, a low water to cementitious materials ratio concrete mixture will be utilized.	Tests, inspections, or a combination of tests and inspections will be performed to ensure the concrete meets the low water to cement ratio limit.	A report concludes that the concrete utilized to construct the as-built Turbine Building below grade concrete foundation and walls have a maximum water to cementitious materials ratio of 0.45.

Table 2.4-11— {Switchgear Building Inspections, Tests, Analyses, and Acceptance Criteria}
(Page 1 of 2)

	Commitment Wording	Inspection, Test, or Analysis	Acceptance Criteria
1	The Switchgear Building is located adjacent to and contiguous with the Turbine Building.	An inspection of the as-built structure will be performed.	The as-built Switchgear Building is located adjacent to and contiguous with the as-built Turbine Building.
2	The Switchgear Building does not impact the ability of any safety-related structure, system, or component to perform its safety function under design basis loads, as specified below. ◆ Normal plant operation (including dead loads, live loads, lateral earth pressure loads, equipment loads, hydrostatic, hydrodynamic, and temperature loads). ◆ Internal events (including internal flood loads, accident pressure loads, accident thermal loads, accident pipe reactions, and pipe break loads, including reaction loads, jet impingement loads, cubicle pressurization loads, and missile impact loads). ◆ External events (including wind, rain, snow, flood, tornado, tornado-generated missiles and earthquake).	An inspection and analysis will be performed to verify the as-built Switchgear Building will withstand design basis loads.	~~A report concludes that the Switchgear Building will not impact the ability of any safety-related structure, system or component to perform its safety function under design basis loads, as specified below. The report also concludes that the design of the Switchgear Building is to the same requirements as a Seismic Category I structure.~~ ◆ Normal plant operation (including dead loads, live loads, lateral earth pressure loads, equipment loads, hydrostatic, hydrodynamic, and temperature loads). ◆ Internal events (including internal flood loads, accident pressure loads, accident thermal loads, accident pipe reactions, and pipe break loads, including reaction loads, jet impingement loads, cubicle pressurization loads, and missile impact loads). ◆ External events (including wind, rain, snow, flood, tornado, tornado-generated missiles and earthquake).
3	The Switchgear Building contains the power supplies and the instrumentation and controls for the Turbine Island, the balance of plant, and the SBO diesel generators.	An inspection of the as-built structure will be performed.	The as-built Switchgear Building houses the power supplies and the instrumentation and controls for the Turbine Island, the balance of plant, and the SBO diesel generators, in accordance with the design.

Insert for ITAAC Table 2.4-11.

Insert for ITAAC Table 2.4-10:

A report concludes that:

a. The Turbine Building will not impact the ability of any safety-related structure, system or component to perform its safety function under design basis loads, as specified below;
b. The design of the Turbine Building is to the same requirements as a Seismic Category I structure; and
c. The as-built separation distance between the Turbine Building and the nearest Seismic Category I structure, system or component is greater than the combined calculated deflections (including effect of settlement) of the Turbine Building and the nearest Seismic Category I structure, system or component, under the design basis loads.

- Normal plant operation (including dead loads, live loads, lateral earth pressure loads, equipment loads, hydrostatic, hydrodynamic, and temperature loads).
- Internal events (including internal flood loads, accident pressure loads, accident thermal loads, accident pipe reactions, and pipe break loads, including reaction loads, jet impingement loads, cubicle pressurization loads, and missile impact loads).
- External events (including wind, rain, snow, flood, tornado, tornado-generated missiles and earthquake).

Insert for ITAAC Table 2.4-11:

A report concludes that:

a. The Switchgear Building will not impact the ability of any safety-related structure, system or component to perform its safety function under design basis loads, as specified below;
b. The design of the Switchgear Building is to the same requirements as a Seismic Category I structure; and
c. The as-built separation distance between the Switchgear Building and the nearest Seismic Category I structure, system or component is greater than the combined calculated building deflections (including effect of settlement) of the Turbine Building and the nearest Seismic Category I structure, system or component, under the design basis loads.

Enclosure 3

Table of Changes to CCNPP Unit 3 COLA Associated with the Response to RAI 315, Question 03.07.02-63, Calvert Cliffs Nuclear Power Plant, Unit 3

Table of Changes to CCNPP Unit 3 COLA

Associated with the Response to RAI No. 315, Question 03.07.02-63

Change ID #	Subsection	Type of Change	Description of Change
Part 2 – FSAR			
GN-10-0174	3.7.2.8	Rewrite of COL Item and response for consistency with U.S. EPR Design Certification Document revision 2 for inclusion in CCNPP Unit 3 COLA revision 7[1].	Revised COL Item for 3.7.2.8 for consistency with U.S. EPR Design Certification Document Revision 2.
CC3-10-0197	3.7.2.8	Rewrite of COL Item and response for consistency with U.S. EPR Design Certification Document revision 2 for inclusion in CCNPP Unit 3 COLA revision 7[1].	Revised COL Item for 3.7.2.8 for consistency with U.S. EPR Design Certification Document Revision 2.
CC3-10-0289	3.7.1, 3.7.2	Rewrite of 3.7 for inclusion in CCNPP Unit 3 COLA revision 7[1].	3.7.1 and 3.7.2 were re-written in response to multiple RAIs for inclusion in CCNPP Unit 3 COLA Revision 7.
CC3-10-0302	3.7.1, 3.7.2	Incorporate COLA markups associated with the response to RAI 253[2].	3.8.1 and 3.7.2 (all of 3.0) were revised to incorporate the revise Ultimate Heat Sink Electrical Building design and respond to RAI 253.
CC3-11-0085	3.7.2	Incorporate COLA markups associated with the response to RAI 253[3].	Added information concerning seismic attriburtes of the Turbine Building, Switchgear Building, and Access Building.

[1] UniStar Nuclear Energy Letter UN#10-300, from Greg Gibson to Document Control Desk, U.S. NRC, of Revision 7 to the Combined License Application for the Calvert Cliffs Nuclear Power Plant, Unit 3, and Application for Withholding of Documents, dated December 20, 2010

[2] UniStar Nuclear Energy Letter UN#10-285, from Greg Gibson to Document Control Desk, U.S. NRC, Ultimate Heat Sink Makeup Water Intake Structure and Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 253, Seismic System Analysis, dated November 16, 2010

[3] UniStar Nuclear Energy Letter UN#11-116, 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 253, Seismic System Analysis, dated March 30, 2011

Change ID #	Subsection	Type of Change	Description of Change
CC3-11-0221	3.7.1.2	Incorporate COLA markups associated with the response to RAI 253[4].	The response to RAI 253 Question 03.07.02-45 involved deleting the term "Seismic Category II-SSE" from the second paragraph in Section 3.7.1.2.
CC3-11-0221	3.7.2.3.3	Incorporate COLA markups associated with the response to RAI 253[4].	The response to RAI 253 Question 03.07.02-45 involved deleting the first sentence from the paragraph in Section 3.7.2.3.3. This sentence discussed Seismic Category II-SSE structures, systems, and components.
CC3-11-0221	3.7.2.8	Incorporate COLA markups associated with the response to RAI 253[4].	The response to RAI 253 Question 03.07.02-45 involved deleting text which referred to the "Seismic Category II-SSE" seismic category in FSAR Section 3.7.2.8 on pages 3-52 and 3-53 of COLA Revision 8. On page 3-53 of COLA Revision 8, a sentence was added to provide information regarding the design of buried fire protection mains.
CC3-12-0093	3.7.2.8	Incorporate COLA markups associated with the response to RAI 334 Question 14.03.02-19[5].	Turbine Island (TI) Structure (Turbine Building and Switchgear Building) design information was added to FSAR Section 3.7.2.8 as part of the RAI 334 Question 14.03.02-19 response.
CC3-12-0216	3.7.2.8	Incorporate COLA markups associated with the response to RAI 367 Question 14.03.02-20[6].	The response to RAI 367 added design information to FSAR 3.7.2.8. This response did not overwrite earlier responses.
CC3-12-0201	3.7.2.8	Acronym Correction[7]	Acronym Correction for SWGB
CC3-13-0101	3.3.2.3	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Building to interaction discussion in FSAR 3.3.2.3.

[4] UniStar Nuclear Energy Letter UN#12-055, from Mark T. Finley to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 253, Seismic System Analysis, dated June 21, 2012
[5] UniStar Nuclear Energy Letter UN#12-038, from Mark T. Finley to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 334, Structural and Systems Engineering - Inspections, Tests, Analyses, and Acceptance Criteria, dated April 20, 2012
[6] UniStar Nuclear Energy Letter UN#12-133, from Mark T. Finley to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 367, Structural and Systems Engineering - Inspections, Tests, Analyses, and Acceptance Criteria, dated November 28, 2012
[7] UniStar Nuclear Energy Letter UN#12-129, from Mark T. Finley to Document Control Desk, U.S. NRC, Acronym Discrepancies, dated November 13, 2012

Change ID #	Subsection	Type of Change	Description of Change
CC3-13-0101	3.7.1, 3.7.1.2, 3.7.1.3.4	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Island structure seismic related discussions to FSAR 3.7.1, 3.7.1.2, and 3.7.1.3.4.
CC3-13-0101	3.7.2.1.3, 3.7.2.2.4, 3.7.2.3.3	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Island structure seismic related discussions to FSAR 3.7.2.1.3, 3.7.2.2.4, and 3.7.2.3.3.
CC3-13-0101	3.7.2.4, 3.7.2.4.1.4, 3.7.2.4.2.4, 3.7.2.4.3.4, 3.7.2.4.4.4, 3.7.2.4.5.4, 3.7.2.4.6.4, 3.7.2.4.7	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Island structure seismic related discussions to FSAR 3.7.2.4, 3.7.2.4.1.4, 3.7.2.4.2.4, 3.7.2.4.3.4, 3.7.2.4.4.4, 3.7.2.4.5.4, 3.7.2.4.6.4, and 3.7.2.4.7.
CC3-13-0101	3.7.2.8	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Modified and added Turbine Island structure seismic related discussions to FSAR 3.7.2.8.
CC3-13-0101	3.7.2.15	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Island structure seismic related discussions to FSAR 3.7.2.15.
CC3-13-0101	Table 3.7-10a	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Island structure seismic related table of values as FSAR Table 3.7-10a.
CC3-13-0101	Table 3.8-4	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Island structure to FSAR Table 3.8-4.

Change ID #	Subsection	Type of Change	Description of Change
CC3-13-0101	Figures 3.7-82 through 3.7-86	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Island structure seismic related Figures 3.7-82 through 3.7-93b.
Part 10 – Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) and ITAAC Closure			
CC3-13-0031	Table 2.4-10 Item 2	Incorporate COLA markups associated with the submittal of CCNPP Unit 3 COLA Revision 9[8].	Revised ITAAC for consistency with U.S. EPR Re-Submittal of Revision 4 of the U.S. EPR Final Safety Analysis Report for Design Certification, dated November 15, 2012[9].
CC3-13-0031	Table 2.4-11 Item 2	Incorporate COLA markups associated with the submittal of CCNPP Unit 3 COLA Revision 9[8].	Revised ITAAC for consistency with U.S. EPR Re-Submittal of Revision 4 of the U.S. EPR Final Safety Analysis Report for Design Certification, dated November 15, 2012.
CC3-13-0101	Table 2.4-10 Item 2	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Turbine Building related seismic requirement to ITAAC Table 2.4-10 Item 2.
CC3-13-0101	Table 2.4-11 Item 2	Incorporate COLA markups associated with the response to RAI 315 Question 03.07.02-63 (this response).	Added Switchgear Building related seismic requirement to ITAAC Table 2.4-10 Item 2.

[8] UniStar Nuclear Energy Letter UN#13-033, from Mark T. Finley to Document Control Desk, U.S. NRC, Submittal of Corrected Revision 9 to the Combined License Application for the Calvert Cliffs Nuclear Power Plant, Unit 3, and Application for Withholding of Documents, dated April 9, 2013

[9] Pedro Salas (AREVA NP Inc.) to Document Control Desk (NRC), "Re-Submittal of Revision 4 of the U.S. EPR Final Safety Analysis Report for Design Certification, NRC:12:057," dated November 15, 2012