ArevaEPRDCPEm Resource

From:	WELLS Russell D (AREVA NP INC) [Russell.Wells@areva.com]
Sent:	Friday, April 17, 2009 10:39 PM
To:	Getachew Tesfaye
Cc:	Pederson Ronda M (AREVA NP INC); BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC)
Subject:	Response to U.S. EPR Design Certification Application RAI No. 130, FSAR Ch 3, Supplement 2 <a e>
Attachments:	RAI 130 Supplement 2 Response US EPR DC.pdf

Getachew,

AREVA NP Inc. (AREVA NP) provided a schedule for the responses to 52 questions of RAI No. 130 on December 08, 2008. AREVA NP submitted Supplement 1 to RAI No. 130 on February 20, 2009 which provided responses to 20 of the 52 questions. The attached file, “RAI 130 Supplement 2 Response US EPR DC-SUNSI.pdf” provides technically correct and complete responses to the remaining 32 questions, as committed.

Since the response file contains security-related sensitive information that should be withheld from public disclosure in accordance with 10 CFR 2.390, a public version is provided with the security-related sensitive information redacted. This email does not contain any security-related information. The unredacted SUNSI version is provided under separate email.

Appended to this file are affected pages of the U.S. EPR Final Safety Analysis Report in redline-strikeout format which support the response to RAI 130 Questions 03.07.01-05, 03.07.01-06, 03.07.01-07, 03.07.01-17, 03.07.02-19, 03.07.02-23, 03.07.02-25, and 03.07.02-27.

The following table indicates the respective pages in the response document, “RAI 130 Supplement 2 Response US EPR DC-SUNSI.pdf,” that contain AREVA NP’s response to the subject questions.

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This concludes the formal AREVA NP response to RAI 130, and there are no questions from this RAI for which AREVA NP has not provided responses.

Sincerely,

(Russ Wells on behalf of)

Ronda Pederson
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From: WELLS Russell D (AREVA NP INC)
Sent: Friday, February 20, 2009 7:31 PM
To: 'Getachew Tesfaye'
Cc: Pederson Ronda M (AREVA NP INC); BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC); SLIVA Dana (EXT); 'John Rycyna'
Subject: Response to U.S. EPR Design Certification Application RAI No. 130, FSAR Ch 3, Supplement 1

Getachew,

AREVA NP Inc. (AREVA NP) provided a schedule for the responses to the 52 questions of RAI No. 130 on December 08, 2008. The attached file, “RAI 130 Supplement 1 Response US EPR DC.pdf” provides technically correct and complete responses to 20 of the 52 questions, as committed.

Appended to this file are affected pages of the U.S. EPR Final Safety Analysis Report in redline-strikeout format which support the response to RAI 130 Questions 03.07.01-12, 03.07.01-15, 03.07.02-2, 03.07.02-10, 03.07.02-17, and 03.07.02-34.

The following table indicates the respective pages in the response document, “RAI 130 Supplement 1 Response US EPR DC.pdf” that contain AREVA NP’s response to the subject questions.

Question #	Start Page	End Page
RAI 130 — 03.07.01-1	2	2
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RAI 130 — 03.07.01-13	8	8
RAI 130 — 03.07.01-14	9	9
RAI 130 — 03.07.01-15	10	11
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The schedule for technically correct and complete responses to the remaining 32 questions is unchanged and provided below:

Question #	**Response Date**
RAI 130 — 03.07.01-5	April 17, 2009
RAI 130 — 03.07.01-6	April 17, 2009
RAI 130 — 03.07.01-7	April 17, 2009
RAI 130 — 03.07.01-8	April 17, 2009
RAI 130 — 03.07.01-10	April 17, 2009
RAI 130 — 03.07.01-11	April 17, 2009
RAI 130 — 03.07.01-17	April 17, 2009
RAI 130 — 03.07.02-1	April 17, 2009
RAI 130 — 03.07.02-3	April 17, 2009
RAI 130 — 03.07.02-4	April 17, 2009
RAI 130 — 03.07.02-5	April 17, 2009
RAI 130 — 03.07.02-6	April 17, 2009
RAI 130 — 03.07.02-7	April 17, 2009
RAI 130 — 03.07.02-9	April 17, 2009
RAI 130 — 03.07.02-11	April 17, 2009
RAI 130 — 03.07.02-12	April 17, 2009
RAI 130 — 03.07.02-13	April 17, 2009
RAI 130 — 03.07.02-14	April 17, 2009
RAI 130 — 03.07.02-15	April 17, 2009
RAI 130 — 03.07.02-16	April 17, 2009
RAI 130 — 03.07.02-18	April 17, 2009
RAI 130 — 03.07.02-19	April 17, 2009
RAI 130 — 03.07.02-20	April 17, 2009
RAI 130 — 03.07.02-21	April 17, 2009
RAI 130 — 03.07.02-23	April 17, 2009
RAI 130 — 03.07.02-25	April 17, 2009
RAI 130 — 03.07.02-26	April 17, 2009
RAI 130 — 03.07.02-27	April 17, 2009

RAI 130 — 03.07.02-28	April 17, 2009
RAI 130 — 03.07.02-29	April 17, 2009
RAI 130 — 03.07.02-30	April 17, 2009
RAI 130 — 03.07.02-31	April 17, 2009

Sincerely,

(Russ Wells on behalf of)
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From: WELLS Russell D (AREVA NP INC)
Sent: Monday, December 08, 2008 7:08 PM
To: 'Getachew Tesfaye'
Cc: 'John Rycyna'; Pederson Ronda M (AREVA NP INC); BENNETT Kathy A (OFR) (AREVA NP INC); DELANO Karen V (AREVA NP INC)
Subject: Response to U.S. EPR Design Certification Application RAI No. 130, FSAR Ch 3

Getachew,

Attached please find AREVA NP Inc.'s response to the subject request for additional information (RAI). The attached file, "RAI 130 Response US EPR DC.pdf" provides technically correct and complete responses to none of the 52 questions.

A complete answer is not provided to any of the 52 questions. The schedule for a technically correct and complete response to these questions is provided below.

Question #	Response Date
RAI 130 — 03.07.01-1	February 20, 2009
RAI 130 — 03.07.01-2	February 20, 2009
RAI 130 — 03.07.01-3	February 20, 2009
RAI 130 — 03.07.01-4	February 20, 2009
RAI 130 — 03.07.01-5	April 17, 2009
RAI 130 — 03.07.01-6	April 17, 2009
RAI 130 — 03.07.01-7	April 17, 2009
RAI 130 — 03.07.01-8	April 17, 2009
RAI 130 — 03.07.01-9	February 20, 2009
RAI 130 — 03.07.01-10	April 17, 2009
RAI 130 — 03.07.01-11	April 17, 2009
RAI 130 — 03.07.01-12	February 20, 2009
RAI 130 — 03.07.01-13	February 20, 2009
RAI 130 — 03.07.01-14	February 20, 2009
RAI 130 — 03.07.01-15	February 20, 2009
RAI 130 — 03.07.01-16	February 20, 2009
RAI 130 — 03.07.01-17	April 17, 2009
RAI 130 — 03.07.01-18	February 20, 2009

RAI 130 — 03.07.02-1	April 17, 2009
RAI 130 — 03.07.02-2	February 20, 2009
RAI 130 — 03.07.02-3	April 17, 2009
RAI 130 — 03.07.02-4	April 17, 2009
RAI 130 — 03.07.02-5	April 17, 2009
RAI 130 — 03.07.02-6	April 17, 2009
RAI 130 — 03.07.02-7	April 17, 2009
RAI 130 — 03.07.02-8	February 20, 2009
RAI 130 — 03.07.02-9	April 17, 2009
RAI 130 — 03.07.02-10	February 20, 2009
RAI 130 — 03.07.02-11	April 17, 2009
RAI 130 — 03.07.02-12	April 17, 2009
RAI 130 — 03.07.02-13	April 17, 2009
RAI 130 — 03.07.02-14	April 17, 2009
RAI 130 — 03.07.02-15	April 17, 2009
RAI 130 — 03.07.02-16	April 17, 2009
RAI 130 — 03.07.02-17	February 20, 2009
RAI 130 — 03.07.02-18	April 17, 2009
RAI 130 — 03.07.02-19	April 17, 2009
RAI 130 — 03.07.02-20	April 17, 2009
RAI 130 — 03.07.02-21	April 17, 2009
RAI 130 — 03.07.02-22	February 20, 2009
RAI 130 — 03.07.02-23	April 17, 2009
RAI 130 — 03.07.02-24	February 20, 2009
RAI 130 — 03.07.02-25	April 17, 2009
RAI 130 — 03.07.02-26	April 17, 2009
RAI 130 — 03.07.02-27	April 17, 2009
RAI 130 — 03.07.02-28	April 17, 2009
RAI 130 — 03.07.02-29	April 17, 2009
RAI 130 — 03.07.02-30	April 17, 2009
RAI 130 — 03.07.02-31	April 17, 2009
RAI 130 — 03.07.02-32	February 20, 2009
RAI 130 — 03.07.02-33	February 20, 2009
RAI 130 — 03.07.02-34	February 20, 2009

Sincerely,

(Russ Wells on behalf of)

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From: Getachew Tesfaye [mailto:Getachew.Tesfaye@nrc.gov]
Sent: Wednesday, November 12, 2008 7:33 PM
To: ZZ-DL-A-USEPR-DL

Cc: Manas Chakravorty; Sujit Samaddar; Michael Miernicki; Joseph Colaccino; John Rycyna
Subject: U.S. EPR Design Certification Application RAI No. 130 (1430,1461),FSAR Ch. 3

Attached please find the subject requests for additional information (RAI). A draft of the RAI was provided to you on October 27, 2008, and on November 12, 2008, you informed us that the RAI is clear and no further clarification is needed. As a result, no change is made to the draft RAI. The schedule we have established for review of your application assumes technically correct and complete responses within 30 days of receipt of RAIs. For any RAIs that cannot be answered within 30 days, it is expected that a date for receipt of this information will be provided to the staff within the 30 day period so that the staff can assess how this information will impact the published schedule.

Thanks,
Getachew Tesfaye
Sr. Project Manager
NRO/DNRL/NARP
(301) 415-3361

Hearing Identifier: AREVA_EPR_DC_RAIs
Email Number: 414

Mail Envelope Properties (1F1CC1BBDC66B842A46CAC03D6B1CD41015EC258)

Subject: Response to U.S. EPR Design Certification Application RAI No. 130, FSAR Ch 3, Supplement 2 <a e>
Sent Date: 4/17/2009 10:38:33 PM
Received Date: 4/17/2009 10:38:42 PM
From: WELLS Russell D (AREVA NP INC)

Created By: Russell.Wells@areva.com

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Tracking Status: None
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MESSAGE	11371	4/17/2009 10:38:42 PM
RAI 130 Supplement 2 Response US EPR DC.pdf		375043

Options
Priority: Standard
Return Notification: No
Reply Requested: No
Sensitivity: Normal
Expiration Date:
Recipients Received:

Response to

Request for Additional Information No. 130 Supplement 2 (1430, 1461), Revision 0

11/12/2008

U. S. EPR Standard Design Certification
AREVA NP Inc.
Docket No. 52-020
SRP Section: 03.07.01 - Seismic Design Parameters
SRP Section: 03.07.02 - Seismic System Analysis
Application FSAR Section: 03.07

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

Question 03.07.01-5:

The SRP 3.7.1 ii SAC 3 states that in addition to the information provided for the supporting media in FSAR Section 3.7.1, the dimensions of the structural foundations, total structural height, design ground water elevation and soil properties such as Poisson's ratio should also be provided. Include in FSAR Section 3.7.1 a reference table which provides the design values used for these parameters.

Response to Question 03.07.01-5:

Dimensions of the structural foundations, total structural height, and design ground water elevation are described in U.S. EPR FSAR Tier 2, Section 3.8 and shown in U.S. EPR FSAR Tier 2, Appendix 3B figures, as referenced in Section 3.7.1.3. Soil properties such as Poisson's ratio are shown in U.S. EPR FSAR Tier 2, Table 3.7.2-9. U.S. EPR FSAR Tier 2, Section 3.7.1.3, will be revised to reference the correct table for soil properties by changing the reference from Table 3.7.2-7 to Table 3.7.2-9.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.7.1.3 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.01-6:

The SRP acceptance criteria 3.7.1 ii SAC 2 states that the material soil damping for foundation soils must be based upon validated values or other data considering variation in the soil properties and strain levels within the soil. In addition, the maximum soil damping value acceptable to the staff is 15 percent per SRP acceptance criteria in SRP3.7.1. In FSAR Table 3.7.1-6, specify the damping values used in the SSI analysis for the corresponding generic soil profiles. Indicate that these values are strain compatible values and do not exceed 15 percent for a damping value.

Response to Question 03.07.01-6:

U.S. EPR FSAR, Tier 2, Table 3.7.2-9 shows damping values used in the soil-structure interaction (SSI) analysis for the corresponding generic profiles. As clarification, Table 3.7.1-6 will be revised to include a note that references Table 3.7.2-9 for applied damping values. Table 3.7.2-9 will also be revised to contain a note stating that shear wave velocities and S-wave damping values are strain-compatible and that damping values do not exceed 15 percent.

U.S. EPR FSAR, Tier 2, Table 3.7.1-6, Note 1 will be revised to clarify that generic soil profile shear wave velocities are strain-compatible.

FSAR Impact:

U.S. EPR FSAR Tier 2, Table 3.7.1-6 and Table 3.7.2-9 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.01-7:

On FSAR Page 3.7-12 last line, the reference Table Number 3.7.2-7 appears not to be correct. Include correct Table Number.

Response to Question 03.07.01-7:

U.S. EPR FSAR Tier 2, Section 3.7.1.3 will be revised to correct the referenced table to Table 3.7.2-9. This change is also addressed in the response to Question 03.07.01-5.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.7.1.3 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.01-8:

Related to FSAR 3.7.1.1.2, the 3.7.1 ii SRP Acceptance Criteria 1B states that “artificial time histories which are not based on seed recorded time histories should not be used.” As such, in Section 3.7.1.1.2 of the FSAR, confirm that the synthetic time histories are based on seed recorded time histories.

Response to Question 03.07.01-8:

Synthetic time histories described in U.S. EPR FSAR Tier 2, Section 3.7.1.1.2 are based on seed recorded time histories in compliance with 3.7.1 ii SRP Acceptance Criteria 1B.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.01-10:

FSAR Table 3.7.1-1 listed the damping value of piping systems in uniform support motion response spectrum analysis as 5 percent. This damping value is different from the value accepted by the staff in RG 1.61, Rev. 1. Technical justification was provided in the AREVA response to RAI, dated November 20th, 2007, on the AREVA NP Piping Analysis Topical Report ANP-10264NP. However, on April 18th, 2008, AREVA provided its second revised response to the RAI of ANP-10264NP, in which AREVA committed to use damping values given in RG 1.61, Rev. 1 for uniform support motion response spectrum analysis. As such, revise Table 3.7.1-1 to confirm the commitment of using the damping value listed in RG 1.61, Rev. 1. In addition, the EXCEPTION in the row “1.61, R1” of the Table 1.9-2 - U.S. EPR Conformance with Regulatory Guides should also be updated to reflect the commitment.

Response to Question 03.07.01-10:

The AREVA NP response to RAI Batch 161, Question 03.12-11, revised U.S. EPR FSAR Tier 2, Table 3.7.1-1 to confirm the commitment of using the damping value listed in RG 1.61, Revision 1. Additionally, U.S. EPR FSAR Tier 2, Table 1.9-2 has been revised to delete the exception to RG 1.61, Rev. 1 and this change will be provided in the next revision to the U.S. EPR FSAR.

FSAR Impact:

U.S. EPR FSAR, Tier 2, Table 1.9-2 will be revised as described in the response.

Question 03.07.01-11:

In Section 3.7.1.2 of the FSAR (first paragraph on pg 3.7-11), it states that in-structure response spectra (ISRS) for the NI common basemat structures are generated using SSE damping values. RG 1.61 requires that the damping values used need to be consistent with the level of stress. Provide the computed stress level (attributed to load combinations using the SSE) for major load carrying members such as walls, columns, floors, etc. of the NI common basemat structures to justify the use of SSE structural damping for the development ISRS.

Response to Question 03.07.01-11:

Calculated stress for critical sections shows that stress levels for combinations using the safe shutdown earthquake (SSE) are consistent with the use of SSE damping and meet RG 1.61 Rev 1 guidelines. Tabulated results for these values are available for review.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.01-17:

FSAR Section 2.5.2.6 indicates that the COL applicant will confirm that the value of shear wave velocity at the bottom of the foundation basemat of the NI common basemat structure is 304 m/sec (1000 ft/sec) or greater. However, similar commitment for other Seismic Category I Structures not located on the common basemat is not provided. According to the acceptance criteria of SRP Section 3.7.1.II.3, potential impact on soil-structure interaction and settlement must be addressed if the minimum shear wave velocity is less than 304m/sec (1000 ft/sec). Initiate appropriate COL information item that requires addressing by a COL applicant the aforementioned potential impact for other Seismic Category I structures not located on the common basemat.

Response to Question 03.07.01-17:

U.S. EPR FSAR Tier 2, Section 2.5.2.6 applies to all Seismic Category I structures, whether or not located on the NI common basemat. If the minimum shear wave velocity is less than 304m/sec (1000 ft/sec), the potential impact on soil-structure interaction and settlement must be addressed by the COL applicant, as described in U.S. EPR FSAR Tier 2, Section 2.5.2.6, Item 7. To clarify this requirement, Item 2 of U.S. EPR FSAR Tier 2, Section 2.5.2.6 will be revised to state that COL applicants will confirm that shear wave velocity for NI basemat and other Seismic Category I structures is 1000 ft/sec or greater, consistent with SRP Section 3.7.1.II.3 acceptance criteria.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 2.5.2.6 will be revised as described in the response and indicated on the enclosed markup.

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Question 03.07.02-1:

In FSAR Section 3.7.2.1.3 (pg 3.7-69, 2nd paragraph), it indicates that the complex frequency response analysis method is used in the seismic SSI analysis of all Seismic Category I structures. AREVA computer code SASSI, Version 4.1B, is used in the SSI analysis of the NI common basemat structures and NAB. Bechtel computer code SASSI 2000, Version 3.1, is used in the SSI analysis of the EPGBs and ESWBs. Describe the differences between these two versions of the SASSI Code, the reason for implementing two versions of the code and provide a comparison of results from a building seismic analysis using each version of the code.

Response to Question 03.07.02-1:

Two different versions of SASSI were used because two different organizations were involved in the SSI analysis of the U.S. EPR structures, with each organization performing work under its respective QA program. Each version of SASSI has been verified and validated in accordance with QA procedures in compliance with Appendix B of 10CFR 50. Regardless of the proprietary manner in which each performs calculations, the two SASSI codes are functionally the same and produce equivalent results with no significant differences.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-3:

Starting on FSAR page 3.7-103, Table 3.7.2-1 through 3.7.2-8 provides information on the modal characteristics of the various stick models included in the SSI model. The frequencies listed indicate frequencies below 50 Hz. FSAR Section 3.7.2.2 (page 3.7-70) indicates that the stick model development was based on comparison of modal responses between the FEMs and the stick models. Considering that the Interim Staff Guidance (COL/DC-ISG-01) indicates that acceptable models, both FEM and equivalent sticks, must be able to capture adequately responses to at least 50 Hz, provide information on the frequency transmission characteristics of the stick models as well as the FEMs used for seismic analysis.

Response to Question 03.07.02-3:

U.S. EPR FSAR Tier 2, Figures 3.7.2-5 through 3.7.2-9 show the refined finite element model (FEM)l used to tune stick model properties. U.S. EPR FSAR Tier 2, Figures 3.7.2-14 through 3.7.2-55 provide spectral comparisons of FEM vs. stick model responses at frequencies up 100 Hz.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-4:

In FSAR Section 3.7.2.3.1 (pg 3.7-71), it indicates that the SASSI SSI model is performed assuming a rigid basemat model. Even with a thick basemat, the flexibility of the mat as well as that of the connecting walls can have an impact on local SSI pressure distributions as well as on moment and shear development in the exterior structural elements. What is the impact of this simplifying assumption on the calculation of seismic design loads, as well as on the generation of in-structure response spectra, particularly at higher frequencies?

Response to Question 03.07.02-4:

A rigid basemat assumption is reasonable given that actual U.S. EPR basemat flexing is analytically insignificant. The reason for this insignificant value is that the NI basemat is over 3-meters thick and is stiffened in the out-of-plane direction by (1) the Containment Building and (2) the Shield Building over the Containment Building (two concentric closed cylinders), (3) exterior walls along the perimeter of the basemat, and, (4) interior bearing walls that are especially substantial due to shielding considerations. These walls provide sufficient stiffening such that global rocking due to soil-structure interaction (SSI) effects of the NI Common Basemat Structures produces rigid body motion of the basemat. Additionally, given in-plane rigidity of the basemat, the overall global response of the structure due to twisting and horizontal translation is also adequately captured using a rigid basemat assumption. The primary response to be addressed with respect to basemat flexibility is potential flexibility in the out-of-plane direction of basemat spans located between the stiffening elements noted above and its effect on vertical seismic response. By evaluating the basemat using methods similar to those used for analysis of flexible floor slabs, the effects of mat flexibility are captured. In this manner, basemat seismic design considers actual basemat flexibility in static analysis of NI Common Basemat Structures.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-5:

In FSAR Section 3.7.2.3.1 (pg 3.7-73), it indicates that the effects of floor and wall flexibilities are not included in the stick models but are accounted for in subsequent analyses following the performance of the modal time history analyses using SDOF models. SRP 3.7.2-SAC-3.C.iii states that local vibration modes should be adequately represented in the dynamic response model. Since what is described in the FSAR is basically a decoupling procedure, provide the basis for this decoupling approach and provide verification that the determination of structural loads and the development of in-structure response spectra are not compromised by the use of this method.

Response to Question 03.07.02-5:

The effect of flexible walls and floors is included in the concrete-only stick model as it is tuned against the concrete-only finite element model.

Additional amplification due to flexible walls and floors response is captured by the finite element model and incorporated into the tuning process.

Guidance from ASCE 4-98 Sections 3.1.7 and 3.1.8 was used in modeling slabs and walls.

From the global response, the spectra generated are used to analyze flexible floors and walls using local models. This process appropriately represents local vibration modes in the dynamic response model, and adequately addresses SRP 3.7.2 Acceptance Criteria.

Also see response to RAI 03.07.02-9.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-6:

In FSAR Section 3.7.2.3.1 (pg 3.7-73), it indicates that adjustments to properties are used to ensure compatibility between the stick models and the FEM results, and indicates that the process provides a “reasonable dynamic compatibility”. The comparisons of in-structure response spectra indicated in the figures attached to this section (Figs. 3.7.2-14 through 3.7.2-55) often show significant differences between spectra peaks as well as frequency shifting between peaks.

A. Provide a basis for the acceptance of the results from the simplified stick models.
B. Address the discrepancy between the stick model ISRS and the FEM ISRS and the impact on the subsequent analysis of supported systems and equipment.
C. SRP 3.7.2-SAC-3.C.ii states that a finite element model must demonstrate that further refinement of the model has only a negligible effect on the solution results. Since the FEM is being used to determine acceptability of the stick models, provide the results of a refinement analysis on the FEMs that meets the acceptance criteria of the SRP and which demonstrate that the FEMs used are adequate to represent the dynamic characteristics of the structure. This discussion should include refinement analyses for the seismic models of the EPGBs and the ESWBs.

Response to Question 03.07.02-6:

ASCE 4-98 provides guidance for the use of simplified models in Section 3.3.1.8.

Properties are adjusted based on the finite element model (FEM) to estimate section properties and mass for the stick model for the first few dominant modes of vibration. Results are compared to establish sufficiency of modeling. Total mass of both models is approximately the same. It is an iterative process to replicate the frequencies and modal masses in the stick model based on the FEM model. Therefore, approximate results are sought, not an exact duplication of behavior.

a) Two corresponding spectral peaks are considered similar when the amplitude of the FEM spectral peak does not exceed the stick model spectral peak by more than 10 percent and the frequency shift remains within 15 percent. Ten percent is the accuracy tolerance of loads, stresses, FS, etc. and 15 percent peak shift is the amount that response spectra are broadened for analysis. All values fall within accuracy limits for these criteria.
b) Refer to response (a). This effect remains within accuracy limitations of the results. Use of multiple soil cases and spectra broadening ensures that modeling uncertainties are addressed.
c) Finite element models were used in Emergency Power Generating Building (EPGB) and Essential Service Water Building (ESWB) seismic analyses. Since stick models were not used, the question regarding dynamic compatibility between stick models and finite element models does not apply to EPGB and ESWB.

Also see response to RAI 130, Question 03.07.02-3 and RAI 108, Question 03.07.03-3.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-7:

In FSAR Section 3.7.2 (pg 3.7-66), it states that the impact of changes to the design during the detailed design phase are evaluated and that the combined deviations are acceptable if the amplitudes of the in-structure response spectra increase by less than 10 percent. Provide the technical basis for these statements to include the impact on code allowables, and provide justification for not performing reanalysis under the conditions described.

Response to Question 03.07.02-7:

As an example of the use of 10 percent criterion, SRP acceptance criterion 3.7.1 ii SAC 1 (page 3.7.2-10 and 3.7.2-12) provides guidance in the TH generation for use in design. It could be 10 percent below the required RS and as much as 30 percent above it.

Technical guidance for these criteria is also found in ASCE 4-98 Section 3.2.2.2.1(f). A 10 percent change is within the accuracy of the calculations, assumptions and properties in the analysis.

Also refer to RAI 3.7.2-6 response (a).

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-9:

In FSAR Section 3.7.2.1.1 (pg 3.7-68), it states that flexible walls and slabs are accounted for by using a modal time history analysis of single degree of freedom oscillators representing the flexible slabs and walls. In Section 3.7.2.3.1 (pg 3.7-73), it describes how the out-of-plane frequency for flexible slabs and walls is determined using either manual methods or by modal analysis using a local FEM model of the floor or wall. There are two issues to be considered with flexible walls and slabs. The first issue is how to determine the amplified structural response and its impact on the seismic design loads. The second issue is how to develop floor ISRS that adequately account for the local structural flexibility. It is not clear from the several descriptions in Section 3.7.2 which methods are used and if the methods described adequately address these two issues. Using examples provide additional detail on the methods of analysis used for flexible slabs and walls and the validity of these methods to address both the response of the flexible structure and the amplified response for supported systems.

Response to Question 03.07.02-9:

See the response to RAI Question 03.07.02-5.

Vertical floor frequencies are determined by independent modal analysis of the finite element model (FEM). These frequencies are used to generate the corresponding single-degree-of-freedom (SDOF) oscillators for the stick models.

From the FEM model, floor response spectra are generated. An SDOF model is generated with the slab properties in the vertical direction considering cracked and uncracked concrete conditions. They are seismically excited and the envelope of responses of both sticks is used for zero period accelerations (ZPA) to be used in the slab design.

For walls, out-of-plane vibrations are evaluated the same way as floor slabs.

Refer to U.S. EPR FSAR Tier 2, Section 3.7.2.9 for additional descriptions.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-11:

In FSAR Section 3.7.2.3.1 (pg 3.7-71), it states that seismic response loads generated include amplified ISRS at representative locations and amplified ISRS at representative flexible slabs. Describe on what basis these representative slabs are selected. Also, for each structure provide the basis and location of response spectra that are developed for both the stick models and the FEMs and identify any locations where response spectra are not generated and the reason for not doing so.

Response to Question 03.07.02-11:

Refer to U.S. EPR FSAR Tier 2, Section 3.7.2.3.1 for description of methodology and location selection criteria. Locations of additional responses will be finalized later in the design process.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-12:

In FSAR Section 3.7.2.3.1 (pg 3.7-78), it describes how the properties were determined for the RCB stick model. Describe to what extent cracking of the concrete in the RCB needs to be considered in seismic modeling and its impact on the dynamic response of this structure.

Response to Question 03.07.02-12:

The Reactor Containment Building (RCB) stick is a prestressed concrete structure. As such, it is anticipated to have minimal cracking so that the use of uncracked section properties is justified.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-13:

In FSAR Section 3.7.2.3.1.2 (pg 3.7-76, 2nd paragraph), it states that tuning of the composite stick model is done by first adjusting the total concrete mass of each individual stick model to correlate with the total mass of the FEM. Provide the basis for an acceptable correlation of mass between the two models and provide some examples of this in your response. Provide the basis for concluding that the FEM properly accounts for the total mass of the structure including dead loads, live loads, snow loads, equipment loads, etc. as specified in the acceptance criteria of SRP 3.7.2-SAC 3.D. If there are other adjustments performed to the stick model, provide a discussion of what these are.

Response to Question 03.07.02-13:

Refer to responses to RAI Questions 03.07.02-5 and 03.07.02-6.

The stick model and finite element model (FEM) concrete masses are within 10 percent of each other.

Stick model tuning is performed using concrete masses only for the FEM and stick models. Masses associated with other dead loads, live loads, precipitation loads, etc. are calculated separately and then added to the stick model for seismic analysis.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-14:

In FSAR Section 3.7.2.4.4 (pg 3.7-86), it states that the NAB is embedded only on its south side and therefore, for the purposes of seismic analysis, it is sufficient to take the NAB as a surface grounded structure. As SRP 3.7.2 requires that embedment effects be considered in an SSI analysis, the basis for this conclusion should be provided.

Response to Question 03.07.02-14:

The Nuclear Island (NI) is considered surface founded in accordance with ASCE 4-98 Section 3.3.4.2.4 (a). The Nuclear Auxiliary Building (NAB) is included in the NI soil-structure interaction (SSI) model. Separation gaps on three sides of the NAB contain engineered backfill. One side is in contact with backfill during a seismic event. Resultant soil pressure on this wall is a factor in wall design. Stiffening effect on overall NAB response is minimal, as indicated in ASCE 4-98 Section 3.3.4.2.4 (a) Commentary.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-15:

In FSAR Section 3.7.2.4.4 (pg 3.7-86), it states that the footprint of the NI model was transformed into an equivalent circle by calculating the radius of the circle that would provide the same area as that of the NI footprint. The computed radius for this circle is 47.70 m (156.52 ft). However, comparing the moment of inertia of the circle to that of the NI footprint will not give equivalent results.

a. Provide an assessment of other equivalent foundation footprints that might have been used and the possible impact of considering only an area equivalency on the results of the SSI analysis.

b. Will the location of the stick model relative to its coordinate location on the actual footprint be different from its coordinate location on the circle? If it is what is the impact on the results of the analysis?

Response to Question 03.07.02-15:

a. Refer to J.E Bowles "Foundation Analysis and Design" 4th Ed. Page 905 paragraph above the Table 20-6 where equivalency is discussed. Independent check indicates that the circle stiffness is close to the actual moment of inertia of the cruciform shape. Additional guidance is provided in ASCE 4-98 Section 3.3.4.2.1 and Commentary.

b. Location of sticks is on the actual geometry.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-16:

In FSAR Section 3.7.2.4.7 (pg 3.7-89), it states that a concrete slab or wall is considered flexible when the frequency of its first out-of-plane frequency mode is less than 40 Hz. The frequency is calculated by assuming un-cracked concrete for section properties. What is the basis for using 40 Hz as the cut-off frequency and why are un-cracked properties assumed in determining whether or not flexibility must be considered in their seismic response? What method is used to determine the out-of-plane frequencies?

Response to Question 03.07.02-16:

The 40 Hz cut-off frequency was used because systems with a first natural frequency of 30 Hz show very low response and are considered rigid. Thus, 40 Hz is conservative and bounding for judgments regarding stiffness of individual elements.

Since normal operating loads for affected slabs and walls do not exceed one-half their ultimate capacity, uncracked properties are normally used.

Conventional dynamic theory is used to determine frequencies of shells and plates.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-18:

In FSAR Section 3.7.2.5 (pg 3.7-90), it describes the development of floor response spectra and states that the ISRS from all SSI analysis cases are enveloped and the envelope is peak broadened by +/- 15 percent to account for uncertainty in structural modeling and SSI analysis. Describe the process that is used to develop the ISRS for flexible walls and floors and provide examples of these in the response.

Response to Question 03.07.02-18:

Refer to response to RAI Question 03.07.02-6.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-19:

In FSAR Section 3.7.2.7 (pg 3.7-95), it addresses the combination of modal responses when the response spectrum method is used and references RG 1.92, Section C. Discuss how the combination of modal responses specifically addresses each of the methods specified in Section C of RG 1.92, Revision 2. FSAR Section 3.7.2.7 describes a method for calculating the effect of the missing mass on seismic analysis results. Discuss and confirm that what is described meets the requirements of RG 1.92 and include such a statement in the FSAR.

Response to Question 03.07.02-19:

The response of the high frequency modes is calculated in accordance with the NRC-approved AREVA NP Topical Report ANP-10264NP-A, "U.S. EPR Piping Analysis and Pipe Support Design," which conforms to Regulatory Guide 1.92, Appendix A guidance for the missing mass considerations.

To add clarification, U.S. EPR FSAR Tier 2, Section 3.7.2.7, will be revised to reference AREVA NP Topical Report ANP-10264NP-A and Regulatory Guide 1.92, Appendix A.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.7.2.7 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.02-20:

In FSAR Section 3.7.2.8 (pg 3.7-95), it discusses the interaction of non seismic with seismic Category I Structures.

a. For the NAB, it states that a reduction in forces is taken for critical structural elements. Are these forces taken from the seismic analysis of the full stick model of the NAB? What is the reduction that is taken?

b. Since the NAB is designed not to collapse on a seismic Category I structure, SRP 3.7.2-SAC-8 states that the non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions, such that the margin of safety is equivalent to that of Category I structure. Describe how the method proposed meets this requirement.

c. Describe the development of the non linear models for the NAB and NI common basemat structures used to determine the potential for seismic interaction and provide the results of the analysis. Identify the elements that are considered to be nonlinear and provide the basis for determining the non-linearity. Since the NAB and NI common basemat structures were analyzed using full stick models, describe why it is now necessary to use a nonlinear analysis employing finite element models with reduced degrees of freedom?

In this same FSAR Section on page 3.7-98, it also states that the NAB shields the NI common basemat structures from collapse of the Radioactive Waste Processing Building (RWPB). However, it does not appear that the NAB is designed to withstand a collapse of the RWPB. Section 3.7.2.8 states that the NAB is designed to allow distortion short of collapse under an SSE event. The basis for stating that the NAB shields the NI common basemat structures from collapse of the RWPB needs to be justified.

Response to Question 03.07.02-20:

a. Appropriate Energy Absorption Factors (F_μ values) were selected from Table 5-1 of ASCE 43-05 (corresponding to LS-A and concrete shear wall structures). These response reduction factors were applied to the seismically induced in-plane forces (shear and tension) and in-plane moments obtained from the seismic analysis of lumped mass stick model. These factors were also applied to the out-of-plane moments acting on walls and floors, obtained either from the lumped mass stick model results or from any subsequent response spectrum analysis for flexible floor panels. It is noted that the discussion of the F_μ values has been provided in U.S. EPR Tier 2, Section 3.7.2.

b. The Nuclear Auxiliary Building (NAB), which is a non-Seismic Category I structure, was designed using ASCE 43-05 for response level corresponding to Limit State (LS) “A” and design ground motion corresponding to the certified seismic design response spectrum. Subject to the associated drift control and detailing requirements per Table 5-2 and Section 6.2.1 of ASCE 43, such design assures that the NAB is collapse-resistant and there is no adverse interaction with the nearby Seismic Category I Nuclear Island (NI) structures.

c. The non-linear models for the NAB and NI Common Basemat structures are described in U.S. EPR FSAR Tier 2, Section 3.7.2.4.2(1). Non-linear elements are at the soil-basemat intersection where springs and dampers are used in the soil-structure interaction (SSI) analysis to calculate the structure stability only.

U.S. EPR FSAR Tier 2, Section 3.7.2.8 makes two statements regarding the NAB shielding NI structures against potential effects from the Radioactive Waste (Processing) Building (RWPB).

> The first statement says, “The NAB is located between the RWPB and the NI Common Basemat Structures and shields the NI Common Basemat Structures from potential interaction.”
>
> The second states, “Therefore, the NAB shields the NI Common Basemat Structures from any adverse effect of collapse of the RWPB.”

The first statement means that the NAB, which is designed against catastrophic collapse during an safe shutdown earthquake (SSE), is situated directly between NI Common Basemat Structures and the RWPB, thus the NAB prevents direct seismic interaction between these structures.

The second statement is a summarization of the foregoing explanation that a postulated seismically induced collapse of the RWPB would be prevented by the intervening NAB structure from affecting NI Common Basemat Structures.

Disregarding presence of the NAB, physical distance between the NI Common Basemat Structures and the RWPB is greater than the height of the RWPB, thus there is no significant potential for a collapse of the RWPB to directly interact with NI Common Basemat Structures.

FSAR Impact

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-21:

Because a number of computer codes are discussed in FSAR Section 3.7.2, the staff is requesting that all computer codes used in the seismic analysis of seismic Category I structures be identified including those used in soil-structure interaction analysis, in developing ISRS, and in determining seismic loads on structures. In addition, descriptions of the programs, program validation, and the extent of application of the programs should be provided. This information should also be included in the FSAR.

Response to Question 03.07.02-21:

Refer to response RAI 155 Supplement 1, Question 03.08.01-13 for the list of computer codes used in the seismic analysis of seismic Category I structures.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-23:

The acceptance criteria of SRP 3.7.2-SAC-3.D states that in addition to the structural mass, equivalent floor load of 243.5 kg/m^2 (50 psf) should be added to represent miscellaneous dead weights and that a mass equivalent to 25 percent of the design live load and 75 percent of the roof design snow load should be included in the dynamic model. In FSAR Section 3.7.2.3-1, the description of the stick models meets the SRP acceptance criteria for live load and snow load, but the provision for the dead load is not addressed. The discussion of the FEM dynamic models does not address any of these additional loads. The additional loads identified in the SRP acceptance criteria should be added to the FSAR and the impact on the results of the seismic analysis should be addressed if these loads were not accounted for in the seismic analysis.

Response to Question 03.07.02-23:

U.S. EPR FSAR Tier 2, Section 3.7.2.3.1 will be revised to add provisions for the dead load in accordance with the acceptance criteria of SRP 3.7.2-SAC-3.D. The same change was made to U.S. EPR FSAR Tier 2, Section 3.7.3.3 to address RAI Batch 187, Question 03.07.03-21.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.7.2.3.1 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.02-25:

In FSAR Section 3.7.2.5, the methods for developing in-structure response spectra (ISRS) are described. It is stated that these follow the guidance of RG 1.122. SRP 3.7.2-SAC-5.C(2) states that guidance of RG 1.122 is augmented by the following: The 3 Hz frequency increment in the last row of RG 1.1.22, Table 1 applies up to the highest frequency of interest. This typically will be the PGA frequency of the design ground response spectrum, which in some cases may significantly exceed 33 Hz. In FSAR 3.7.2.5 on page 3.7-90, there is a table that shows the frequency increment for sets of frequency ranges at which ISRS acceleration values are computed. From 22 Hz to 40 Hz, the frequency increment is 3 Hz which agrees with the RG. From 40 Hz to 50 Hz, a frequency increment has not been provided. From 50 Hz to 100 Hz, a frequency interval of 50 Hz is indicated. The table should be revised to meet the guidance provided in the SRP acceptance criteria for a 3 Hz increment up to the highest frequency of interest and to also add the frequency increment used from 40 to 50 Hz. Also, the highest frequency of interest should be indicated and the basis for its selection should be provided. FSAR Section 3.7.2.5(2) (pg 3.7-91) describes the development of response spectra for the EPGB and ESWB. It states that response spectra are calculated at a total of 241 frequencies from .2 to 50 Hz with 100 frequencies per decade that are uniformly spaced in the log scale. A table of frequency increments and frequency ranges should be provided similar to that provided for the NI common basemat structures on page 3.7-90 and if different than the requirements of RG 1.122 those differences should be justified.

Response to Question 03.07.02-25:

- The 22 to 40 Hz range will be revised to a 22 to 50 Hz range in U.S. EPR FSAR Tier 2, Section 3.7.2.5.
- The highest frequency of interest is for the U.S. EPR is 40 Hz as approved by the NRC on AREVA NP Topical Report ANP-10264NP-A, “U.S. EPR Piping Analysis and Pipe Support Design” on page 10, second paragraph, last sentence.
- For Emergency Power Generating Building (EPGB) and Essential Service Water Building (ESWB), ISRS are calculated at a total of 241 frequency points between 0.2 and 50 Hz with 100 frequencies per frequency decade that are uniformly spaced in the log scale. This is equivalent to approximate increment of 2.33 percent between all adjacent frequency points within this range. This frequency increment is smaller than that required by RG 1.122 and thus, satisfies this RG requirement.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.7.2.5 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.02-26:

In FSAR Section 3.7.2.6(1) (pg 3.7-94) for the NI common basemat structures, it states that for member forces and moments, the STRESS module of the SASSI code outputs the maximum member force/moment due to each component of earthquake motion. It further states that these member forces and moments are combined by the SRSS. This effectively eliminates the sign of the force (compression or tension) and of the bending moment (positive or negative). In concrete design, the sign or direction of a force or moment is important in properly sizing the member and in determining the correct amount of reinforcement. The amount of shear reinforcement that is required will also be affected by the direction of the axial force and bending moment. Thus, the staff is asking how the method described in the section of the FSAR properly accounts for the sign of the force or moment and how this is used in the design of concrete members. In addition, describe how the multiple sets of input motion time histories are accounted for in determining the maximum member forces and moments. As there are twelve cases analyzed for the NI structures, how the maximum design values are determined should also be described.

Response to Question 03.07.02-26:

Refer to U.S. EPR FSAR Tier 2, Section 3.8.4.4.1. Note that seismic acceleration (zero period acceleration) is used for the detail design. The forces from the static analysis have a positive or negative sign and the worst case combination is used in the design. The combination of loads is also discussed in this section.

U.S. EPR FSAR Tier 2, Section 3.7.2.4.6 discusses envelope of cases analyzed and maximum values selection

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-27:

The last paragraph FSAR Section 3.7.2.4.6(2) (Page 3.7-89) indicates that subsequent analysis will incorporate certain design details for the EPGBs and ESWBs that are not reflected in the existing respective SASSI model used for SSI analyses described in FSAR Section 3.7.2. The design details not yet included are discussed in FSAR Section 3.8.4.4.3 for the EPGBs and FSAR Section 3.8.4.4.4 for the ESWBs. As the effects of these details have not been included in the application for design certification, establish a COL information item to address this issue.

Response to Question 03.07.02-27:

This comment pertains to two different issues for Emergency Power Generating Building (EPGB) and Essential Service Water Building (ESWB). The following are the responses.

EPGB

As noted in U.S. EPR FSAR Tier 2, Section 3.8.4.4.3, it is acknowledged that some of the information related to mechanical items such as layout of small floor openings, weights of miscellaneous equipment, etc. at Elevation 51 ft-6 inches slab included in the SASSI finite element model may be preliminary. However, this is not expected to have any significant effect on either the SASSI results or the structural design. The information used will be confirmed and updated during the detailed design phase as a part of the generic design certification of the U.S. EPR and the U.S. EPR FSAR will be updated accordingly. Therefore, this is not a COL information item.

ESWB

As noted in U.S. EPR FSAR Tier 2, Section 3E.3.1, ten feet extension of the basement slab beyond the building walls was not included in the soil-structure interaction (SSI) analysis. This was added during the preliminary design to resist overturning moment. Subsequent analysis performed has demonstrated that added slab extension is not required for stability. The U.S. EPR FSAR will be revised to reflect deletion of the slab extension.

Since SSI analysis and structural design are compatible with the generic design of the U.S. EPR, there is no need to establish a COL information item.

Additional/revised information is added for clarity.

U.S. EPR FSAR Tier 2, Section 3.7.2.4.6, Section 3.8.4.4.4, Section 3.8.5.1.3, Appendix 3E.3, Appendix 3E.3.1,Table 3E.3.1, Figures 3.8-95, 3.8-101, 3.8-102, 3B-69, 3B-75, 3B-76, 3E.3.2, 3E.3-3 and 3E.3-7 will be revised as indicated on the enclosed markup. Figure 3E.3-10 will be added to Appendix 3E.

FSAR Impact:

U.S. EPR FSAR Tier 2, Section 3.7.2.4.6, Section 3.8.4.4.4, Section 3.8.5.1.3, and Appendix 3E will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.02-28:

In FSAR Section 3.7.2.8 (Third Bullet), it is stated that “conventional seismic structures that have the potential to interact with Seismic Category I structures are assessed for collapse potential under SSE and tornado loading (acting independently). Seismic demand for the SSE is computed in accordance with ASCE 4-98, Reference 1 and the methodologies in Section 3.7.2. Seismic load combinations are developed in accordance with ASCE 43-05 ….” ASCE 4-98 has not been accepted by the Staff as a guidance document. In addition, SRP acceptance criteria 3.7.2 –SAC-8C requires that “the non-Category I structure will be analyzed and designed to prevent its failure under SSE conditions, such that the margin of safety is equivalent to that of Category I structures.” As such, in addition to the NAB which was addressed in RAI 3.7.2-20, demonstrate that non-Category I structures (not analyzed and designed as seismic Category II structures) having the potential of interaction with Category I structures will not slide or overturn during a SSE level earthquake and will have the margin of safety equivalent to that of Category I structures as stated by the acceptance criteria of SRP-SAC-8.C. Address how the NAB, Access Building, Turbine Building, Radioactive Waste Processing Building (RWPB), Fire Protection Storage Tanks and Buildings satisfy the SRP guidance.

Response to Question 03.07.02-28:

Non-Category I structures other than the Nuclear Auxiliary Building (NAB) are evaluated for potential interaction with Category I structures using the guidelines listed in response to RAI 03.07.02-20.

Seismic design interaction based on parameters used to establish adequacy of structural separations using safe shutdown earthquake (SSE) excitations is shown in U.S. EPR FSAR Tier 2, Table 3.7.2-29, which conforms to SRP guidance.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-29:

In FSAR Section 3.7.2.8 (fourth bullet), it is stated that “for Conventional Seismic structures that have the potential to interact with Seismic Category I structures, the combined seismic deflection is less than the separation distance (i.e., gap) between the structures.” Calculation of the combined seismic deflection involves seismic analyses of both the Category I structure in question, as well as the conventional seismic structures having the potential to interact with the Category I structure. While the Category I structure is analyzed and designed to seismic Category I requirements, the conventional seismic structures may not be analyzed to Category I seismic requirements. Accordingly, confirm that seismic analysis of conventional seismic structures that have the potential to interact with Category I structures is based on the same criteria as seismic Category I structures.

Response to Question 03.07.02-29:

See the response to RAI Questions 3.7.2-27 and 3.7.2-28.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-30:

In FSAR Section 3.7.2.3.1.2(2), the second paragraph of this section states that “a single-mass rigid stick, of which the base is connected to the main stick at elevation 37.6 m (123 ft, 4-1/4 in) where the crane rail is located, is used to represent the polar crane.” This implies that the polar crane subsystem in the parked position has been considered to be rigid compared to the supporting system, and also to be rigidly connected to the supporting system. A parametric study was performed (refer to second paragraph on FSAR Page 3.7-79) to verify sufficiency of this representation by comparing response spectra generated from the stick model where the crane is represented by a single rigid mass with the corresponding spectra generated from a modified stick model in which the rigid single-mass stick for the crane assembly is replaced by a flexible one. However, the parametric study considered only one condition where the crane assembly is assumed to have a resonant frequency coincident with the fundamental frequency of the containment. No additional parametric studies covering other potential frequency ranges for the crane assembly were provided to demonstrate that the selected condition for the parametric study is conservative. As such, provide additional justification to demonstrate that the parametric study performed will envelop all potential frequencies of the polar crane assembly (consisting of Crane Rail, Crane Bridge, Trolleys, etc) in the parked position.

Response to Question 03.07.02-30:

Seven cases were studied in which the flexibility of the crane model was varied. The response was monitored at the attachment to the Containment and at the Containment dome. The rigid model response case bounds the other cases.

For seismic analysis, the polar crane is parked at the farthest point from centers of mass and rigidity to provide the largest eccentricity and therefore the greatest contribution to design loads.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

Question 03.07.02-31:

In FSAR 3.7.2.1.1, the last paragraph of this section states that “As a general rule, the value for the maximum time step is no larger than one-fifth of the lowest natural period of interest.” However, for most of the commonly used integration methods, the maximum time step is limited to one-tenth of the smallest period of interest, which is generally the reciprocal of the cutoff frequency. In addition, in accordance with industry practice and as described in Section 3.2.2.1(c) of ASCE 4-98, an acceptable approach for selecting the actual time step (Δt) is that the Δt used shall be small enough such that the use of one-half of Δt does not change the response by more than 10 percent. As such, the staff requests AREVA provide a technical justification for not considering common industry practices in this regard.

Response to Question 03.07.02-31:

See the response to RAI 108 Question 3.7.3-4.

FSAR Impact:

The U.S. EPR FSAR will not be changed as a result of this question.

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soil-structure interaction analyses performed for the U.S. EPR in addressing the following evaluation guidelines.

1. The applicant will confirm that the peak ground acceleration for the GMRS is less than 0.3g.

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2. The applicant will confirm that the low-strain, best-estimate, value of shear wave velocity at the bottom of the foundation basemat of the NI Common Basemat Structures and other Seismic Category I structures is 1000 fps, or greater. This comparison will confirm that the NI Common Basemat Structures and other Seismic Category I structures are founded on competent material.

3. The applicant will demonstrate that the FIRS are enveloped by the CSDRS for the U.S. EPR using the guidance provided in Section 3.7.1.1.1.

4. The applicant will demonstrate that the site-specific profile is laterally uniform by confirming that individual layers with the profile have an angle of dip no greater than 20 degrees.

5. The applicant will demonstrate that the idealized site soil profile is similar to or bounded by the 10 generic soil profiles used for the U.S. EPR. The 10 generic profiles include a range of uniform and layered site conditions. The applicant also considers the assumptions used in the SSI analyses, as described in Section 3.7.1 and Section 3.7.2.

6. If the conditions of steps one through five are met, the characteristics of the site fall within the site parameters for the U.S. EPR and the site is acceptable.

7. If the conditions of steps one through five are not met, the applicant will demonstrate by other appropriate means that the U.S. EPR is acceptable at the proposed site. The applicant may perform intermediate-level additional studies to demonstrate that the particular site is bounded by the design of the U.S. EPR. An example of such studies is to show that the site-specific motion at top-of-basemat level, with consideration of the range of structural frequencies involved, is bounded by the U.S. EPR design.

8. If the evaluations of step 7 are not sufficient, the applicant will perform detailed site-specific SSI analyses for the particular site. This site-specific evaluation will include dynamic seismic analyses and development of in-structure response spectra (ISRS) for comparison with ISRS for the U.S. EPR. These analyses will be performed in accordance with the methodologies described in Section 3.7.1 and Section 3.7.2. Results from this comparison will be acceptable if the amplitude of the site-specific ISRS do not exceed the ISRS for the U.S. EPR by greater than 10 percent on a location-by-location basis. Comparisons will be made at the following key locations, defined in Section 3.7.2:

    A. Reactor Building Internal Structures (RBIS)—Reactor Vessel Support at elevation +16 ft, 10-3/4 in (Figures 3.7.2-74, 3.7.2-75, and 3.7.2-76) and steam generator supports at elevation +63 ft, 11-3/4 in (Figures 3.7.2-77, 3.7.2-78, and 3.7.2-79).

affects of SSSI, as described above in Section 3.7.1.1.1. Seismic SSI analyses are described in Section 3.7.2.4.

Table 3.7.1-6 shows the soil layering, the assumed strain-dependent properties, and the EUR design control motion associated with the 12 analysis cases. The variation in shear wave velocity in each of the assumed profiles is illustrated in Figure 3.7.1-31—U.S. EPR Standard Plant Generic Soil Profiles - Shear Wave Velocity for SSI Analysis Cases, and Figure 3.7.1-32—U.S. EPR Standard Plant Generic Soil Profiles - Shear Wave Velocity for SSI Analysis Cases. Section 3.7.2.4.1 notes that, for SSI analysis for U.S. EPR design certification, the assumed generic shear wave velocities are taken to be strain-compatible values during seismic events, i.e., assumed relationships to depict the strain-dependent modulus-reduction and hysteretic damping properties are not used.

Soil density is varied to correspond with the assumed generic site conditions associated with the three EUR control motions; for example, the SSI model for an analysis case that involves a control motion for a soft site includes lower soil density in the generic profiles than a model for a control motion for a hard soil site. Soil density variations also account for the assumed material variation within a profile. Soil densities in the SSI analysis vary from 110 to ~~125~~156 pcf for soil. Material damping values for soil vary from 1 to 7 percent, with 1 percent damping used for stiffer soils and 7 percent for softer soils. The soil material damping ratio for compression wave propagation (β_p) is conservatively taken to be one-third of the shear wave propagation damping ratio. The maximum material damping value for soil does not exceed 15 percent. The soil properties associated with the various shear wave velocities assumed in the 10 generic soil profiles are discussed further in Section 3.7.2.4.1 and summarized in ~~Table 3.7.2-7~~Table 3.7.2-9.

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Details of the site response and SSI analyses are provided in Section 3.7.2.4. Section 2.5 addresses the geologic, seismologic, and geotechnical requirements necessary to confirm that conditions for a specific site are enveloped by the generic soil profiles used to design the standard plant.

3.7.1.4 References

1. European Utility Requirements for LWR Nuclear Power, Volume 2, Revision C, April 2001.

2. ASCE/SEI 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,” American Society of Civil Engineers, 2005.

3. Report 1053-21.1-4, "Cable Tray and Conduit Raceway Seismic Test Program, Release 4," ~~Report 1053-21.1-4,~~ Bechtel-ANCO Engineers, Inc., December 15, 1978.

Table 3.7.1-6—Generic Soil Profiles for the U.S. EPR Standard Plant

Soil Case No.	Seismic Control Motion Applied[2]	Soil Profile (Half-space or Layered)	Shear-wave Velocity of Soil[1]
1u	EUR Soft	Half-space	700 ft/s
2u	EUR Soft and Medium	Half-space	1640 ft/s
3u	EUR Medium	Half-space	2625 ft/s
4u	EUR Medium and Hard	Half-space	3937 ft/s
5u	EUR Hard	Half-space	5249 ft/s
5a	EUR Hard	Half-space	13,123 ft/s
1n2u	EUR Soft	Linear gradient within a 100 ft layer over a half-space	820 to 1640 ft/s
2sn4u	EUR Medium	49 ft uniform layer over a half-space	1640/3937 ft/s
2n3u	EUR Medium	Linear gradient within a 200 ft layer over a half-space	1640 to 2625 ft/s
3r3u	EUR Medium	20 ft uniform layer over 33 ft stiffer layer followed by soil half-space	2625/5249/2625 ft/s

Notes:

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1. Shear wave velocities of generic soil profiles are ~~used to define~~ strain-compatible~~-properties~~.
2. For the EPGB and ESWB, the modified CSDRS is used for all soil profiles.
3. See Table 3.7.2-9 for damping valves used.

The stick models are developed by first locating key elevations (typically the major floor slab elevations) in the structure. Between two successive key elevations, two vertical massless sticks are developed. One stick is located at the center of shear area and the other at the center of axial area respectively, of the vertical structural elements between the two given key elevations. Section properties of the two sticks are determined by hand calculations based on the structural drawings. The total axial area of the vertical structural elements is assigned to the stick located at the center of axial area. The remaining five section properties, including the total shear areas along the two global axes and the total moments of inertia about the three global axes, are assigned to the stick located at the center of shear area. The two sticks are connected to each other at both their upper and lower ends with a horizontal rigid beam. For the NI stick models, no structural credit is taken for the stiffness of the steel liner plate in both the reactor containment and the spent fuel pool.

At the key elevations of the structure, a lumped mass is placed at the center of mass. The lumped mass is connected with horizontal rigid beams to the center of shear area and center of axial area located at the same elevation. It includes mass contributions from the following elements:

- Floor or roof slab(s), when applicable, at the particular elevation.
- Walls and miscellaneous floor slabs and platforms (including platform live load) within half height to the next key elevation below.
- Walls and miscellaneous floor slabs and platforms (including platform live load) within half height to the next key elevation above.
- Permanent equipment and distribution systems supported by slabs and platforms.
- Water in pools under normal operating conditions.

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- Twenty-five percent of the live loads (variable loads) on floor slabs and platforms.
- Seventy-five percent of the maximum snow load on roof slabs.
- Miscellaneous dead loads of at least 50 psf.

The total mass of water in a pool during normal operating conditions is lumped at the bottom slab of the pool in the vertical direction. In the horizontal direction, the mass of the water is distributed to the nodes along the height of the pool using tributary areas. For the purpose of the stick model, water mass is considered as a permanent load if present during normal operating conditions. The frequency of water sloshing is typically low compared to the first horizontal mode frequency of the structure housing the pool. As such, water sloshing has a negligible effect on the global seismic response of the structure and hence may be ignored in the development of the stick model. The effect of water sloshing however, is considered in the local analysis and detailed design

Case Inter-Story Forces and Moments in Reactor Building Internal Structures to Table 3.7.2-25—Worst Case Inter-Story Forces and Moments in Safeguard Building 2/3 Shield Structure show the worst case inter-story forces and moments in the members of the individual sticks for the NI Common Basemat Structures.

The time history of the displacement at the NI Common Basemat relative to the input ground motion is determined by double integrating the acceleration response time history at the basemat Node 417, applying a linear baseline correction, and subtracting from it the displacement time history of the free field ground motion for each SSI analysis case. Table 3.7.2-26—Maximum NI Common Basemat Displacement Relative to Free Field Input Motion lists the peak relative displacement at Node 417 for all twelve SSI analysis cases. The maximum relative displacement at a given structural location in the NI Common Basemat Structures with respect to the basemat is conservatively taken from the equivalent static analysis of the FEM of the NI Common Basemat Structures described in Section 3.8.4.

(2) EPGB and ESWB

From the nodal acceleration response time histories generated from the SSI analysis, maximum nodal accelerations in each given global direction and due to each of the three components of the input ground motion are extracted. For each of the ten generic soil cases, the extracted maximum nodal accelerations are used to compute the weighted averaged maximum nodal accelerations in each direction and due to each ground motion component at each given elevation for the entire floor or for different regions on the floor. The weighting factors used in the averaging process are the applicable nodal masses. In each direction, the averaged maximum nodal accelerations due to the three ground motion components are then combined using the (1.0, 0.4, 0.4) factor rule to determine the combined average maximum nodal acceleration in the given direction. These~~Such~~ maximum nodal accelerations form the basis of ~~represent~~ the seismic loads used in the equivalent static analysis of the ~~structure~~structural design. Table 3.7.2-27—Worst Case Maximum Accelerations in EPGB and Figure 3.7.2-28—Worst Case Maximum Accelerations in ESWB show the worst case maximum ZPAs at different elevations of the EPGB and ESWB, respectively.

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As discussed in Section 3.8.4.4.3~~ and Section 3.8.4.4.4, ~~, subsequent analyses will incorporate certain design details for the EPGBs and ESWBs that are not reflected in the existing respective SASSI models used for the SSI analyses described in Section 3.7.2. The subsequent analyses will determine the impact of these design details on the seismic responses and ISRS presented in Section 3.7.2.

computed. The ISRS are calculated using AREVA code RESPEC, Version 1.1A, at the following 79 frequencies:

Frequency Range (Hz)	Frequency Increment (Hz)
0.2 to 3.0	0.10
3.0 to 3.6	0.15
3.6 to 5.0	0.20
5.0 to 8.0	0.25
8.0 to 15.0	0.50
15.0 to 18.0	1.00
18.0 to 22.0	2.00
03.07.02-25 22.0 to ~~40~~50.0	3.00
50 to 100	50.0

The above frequencies for ISRS generation comply with the guidelines set forth in Table 3.7.1-1 of SRP Section 3.7.1 in Reference 2. At each given structural elevation along the stick models, ISRS at the lumped mass point and building corner nodes (typically four corner nodes) are calculated for each SSI analysis case. The envelope of the ISRS at these locations represents the ISRS at the particular structural elevation for the SSI particular analysis case. The ISRS from the twelve SSI analysis cases are enveloped, and the spectrum envelope is broadened by ±15 percent and smoothed to account for uncertainty anticipated in the structural modeling and SSI analysis techniques.

(2) EPGB and ESWB

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Response spectra are calculated using Bechtel computer code SASSI 2000, Version 3.1, at a total of 241 frequencies from 0.2 to 50 Hz, with 100 frequencies per decade that are uniformly spaced in the log scale. This is equivalent to a frequency increment of approximately 2.33 percent between all adjacent frequency points. At each given direction and location in the structural model, response spectra are first computed separately for the floor acceleration response time histories due to the three components of input ground motion. The three resulting response spectra are then combined using the SRSS method to produce the ISRS in the corresponding direction and at the given structural location. The ISRS from all ten generic soil cases are then enveloped, and the ISRS envelope is broadened by ±15 percent and smoothed to account for uncertainty anticipated in the structural modeling and SSI analysis techniques.

Results of the Response Spectrum Development

The results of the response spectrum development are presented below for the NI Common Basemat Structures, EPGB and ESWB separately:

(1) NI Common Basemat Structures

Figure 3.7.2-68—Response Spectra at NI Common Basemat Bottom Node 417 - 5% Damping X-Direction, Figure 3.7.2-69—Response Spectra at NI Common Basemat

3.7.2.6 Three Components of Earthquake Motion

(1) NI Common Basemat Structures and NAB

As previously stated in Section 3.7.2.4.6, the floor acceleration time history in a given direction is obtained by algebraically combining the three corresponding time histories due to the three earthquake components. Therefore, both the floor ZPA and the ISRS for the floor acceleration time history properly account for the contributions from the three components of earthquake motion. For member forces and moments in the stick models, the STRESS module of SASSI code outputs the maximum member force/moment in the stick model due to each earthquake motion component. The maximum member forces/moments due to the three earthquake motion components are then combined by the SRSS rule to obtain the maximum total member force/moment. The use of the SRSS rule is consistent with the guidelines specified in RG 1.92, Revision 2.

(2) EPGB and ESWB

As previously stated in Section 3.7.2.4.6, the ZPA of the floor acceleration time histories in a given direction due to the three earthquake motion components are combined using the (1.0, 0.4, 0.4) rule. The response spectra for the floor acceleration time histories in a given direction due to the three earthquake motion components are combined using the SRSS rule to determine the combined ISRS. The (1.0, 0.4, 0.4) rule is also consistent with the guidelines specified in RG 1.92, Revision 2.

3.7.2.7 Combination of Modal Responses

When the response spectrum method of analysis is used, the maximum modal responses are combined using one of the methods specified in RG 1.92, Section C, Revision 2. Such combination methods include the grouping method, ten percent method and double sum methods, and they consider the effects of closely spaced modes having frequencies differing from each other by 10 percent or less of the lower frequency.

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The effect of missing mass for modes not included in the analysis is accounted for by calculating the residual seismic load in accordance with AREVA NP Topical Report ANP-10264NP-A (Reference 11) and RG 1.92, Appendix A, Revision 2.~~equal to the ZPA on the input response spectrum times the missing mass. The residual seismic load is added to the combined modal response determined from the response spectrum method of analysis.~~

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

Figure 1.2-1 and Figure 3B-1 show the layout of structures for a typical U.S. EPR standard plant. The Access Building and Turbine Building are site-specific structures. A COL applicant that references the U.S. EPR design certification will provide the site-

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

6. ACI 318-05, "Building Code Requirements for Structural Concrete and Commentary," American Concrete Institute, 2005.

7. ANSI/AISC 341, "Seismic Provisions for Structural Steel Buildings," American National Standards Institute/American Institute of Steel Construction, 2005.

8. ANSI/AISC 360, "Specifications for Structural Steel Buildings," American National Standards Institute/American Institute of Steel Construction, 2005.

9. ASCE Standard 7-05, "Minimum Design Loads for Buildings and Other Structures," Appendix 11A, "Quality Assurance Provisions," American Society of Civil Engineers, January 1, 2006.

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10. R. Kennedy and F. Ostadan, "Consistent Site-Response," Workshop on Seismic Issues: Consistent Site-Response/Soil-Structure Interaction Calculations, at EPRI Palo Alto, California, September 25-26, 2008, (ADAMS Accession No. 082550165).

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

Table 3.7.2-9—Soil Properties Associated With Different Generic Shear Wave Velocities

Shear Wave Velocity (ft/s)	Shear Wave Velocity (m/s)	Poisson's Ratio μ	Weight Density (pcf)	Weight Density (kN/m3)	S-Wave Damping (%)
700	213	0.40	110	17.28	7
820	250	0.40	110	17.28	7
1640	500	0.40	110	17.28	4
2625	800	0.40	115	18.07	2
3937	1200	0.40	120	18.85	1
5249	1600	0.40	125	19.64	1
13,123	4000	0.35	156	24.51	1

Notes:

1. P-wave damping is taken to be 1/3*S-wave damping.
2. When shear wave velocity varies linearly in a layer, other properties vary accordingly.
3. P-wave velocity = S-wave velocity*$[2(1-\mu)/(1-2\mu)]^{1/2}$.
4. Shear-wave velocities and S-wave damping values are strain compatible. Damping values do not exceed 15 percent.

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spectra. Therefore, a subsequent analysis will be performed with these details in the Finite Element Model to confirm the seismic responses and in-structure response spectra presented in Section 3.7.2. The design of the EPGBs will conform to the structural acceptance criteria described in Section 3.8.4.5.

3.8.4.4.4 Essential Service Water Buildings

Reinforced concrete elements for the four ESWBs consist of slabs, beams, shear walls, and foundation basemat to transfer imposed loads to the supporting soil. Structural steel framing is used to support the missile barriers protecting the safety-related fans.

Similar to the EPGBs, the ESWBs are analyzed and designed using a 3D finite element model representing the structure. The finite element model is generated using the GT STRUDL computer code. The use of the model for both static and dynamic analyses, including extraction of results for design, is almost identical to the methods presented in Section 3.8.4.4.3. Similarly, the GT STRUDL model is used to provide an accurate representation of the structure for translation to an SSI model (SASSI 2000) for seismic analysis. As such, only model variations are addressed below.

In addition to structural dead loads, slab live loads, piping loads and equipment loads, the GT STRUDL finite element model for the ESWBs includes the weight of non-structural fill, hydrostatic loads, hydrodynamic loads, and soil pressures (including surcharge pressures). The appropriate accelerations from the SSI analysis are applied to the tributary floor areas and walls to obtain the equivalent static seismic loads.

Dead load, live load, equipment loads, and piping loads are combined with the equivalent static seismic loads for structural design in accordance with the provisions of ACI 349-01, with supplemental guidance of RG 1.142, ACI 350-06, and ACI 350.3-06. The evaluation of walls and slabs for external hazards (e.g., tornado generated missiles) is performed by local analyses, including ductility evaluations. The elastic solution methodology of ASCE 4-98 is used for the dynamic soil pressures associated with the 22 feet embedment of the ESWBs.

Seismic induced lateral soil pressure on below grade walls are evaluated considering the following cases:

- The seismic soil pressure as equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98, Section 3.5.3.2.
- The seismic soil pressure as equal to the passive earth pressure.

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Additional information on the seismic analysis approach for the ESWBs is contained in Section 3.7.2.

~~For the design of the ESWBs, the foundation basemat extension, as explained in the ESWB section of Appendix 3E, is not reflected in the SASSI Finite Element Model used~~

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~~for SSI analyses. Therefore, a subsequent analysis will be performed with these added details in the Finite Element Model to confirm the foundation basemat design. The design of the ESWBs will conform to the structural acceptance criteria described in Section 3.8.4.5.~~

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

The design of buried conduit and duct banks, and buried pipe and pipe ducts is site-specific. Buried Seismic Category I conduit, electrical duct banks, pipe, and pipe ducts will be analyzed and designed in accordance with the specific requirements of the systems. In addition, these items will be designed for the effects of soil overburden, surcharge, groundwater, flood, seismic soil interaction, and other effects of burial. Concrete components of buried items will be designed in accordance with ACI 349-2001, including the exceptions specified in RG 1.142. Steel components of buried items will be designed in accordance with ANSI/AISC N690-1994 (R2004), including Supplement 2.

Static and long-term analyses of buried items will be based on soil properties under consolidated drained conditions of the soil. Buried items will be designed for soil loads corresponding to the weight of the overlying soil prism.

Live loads will be applied, such as those imposed by truck and rail traffic and by construction equipment and activities. Impact factors will be applied to live loads as appropriate. Where buried items are vulnerable to highway or railway traffic loads, the potential for fatigue-induced failure will be evaluated. The minimum burial depth of buried items will conform to guidance from ASME 31.4 and ASME 31.8.

Buried items will be designed for freeze-thaw induced stresses and for other thermally induced stresses due to soil and ambient air temperatures. Interfacial longitudinal friction effects will be evaluated for buried pipes that are encased in larger pipes or in concrete trenches or boxes.

In cases where buried items are located below the ground water table or where seasonal change in the ground water table is significant, the effect of buoyancy and the increased weight of water will be evaluated. These evaluations will include the effects of fluctuations in ground water level and the effects of flood.

Seismic load effects on buried items will be evaluated using dynamic analyses or equivalent static load methods. For seismic-related and dynamic analyses, the shear strength of soil will be based on the consolidated-undrained triaxial stress conditions of the soil. The procedure for evaluating the structural integrity of buried items under seismic conditions will involve determination of the axial and bending strains in the system due to seismic wave propagation in the surrounding soil mass. The axial force and bending stresses will be computed using the buried item material properties (e.g., Young's modulus and pipe section modulus). Pipe ovalization will be computed based

Horizontal shear loads are transferred from the NI Common Basemat Structure foundation basemat to the underlying soil by friction between the bottom of the basemat, mud mat (or both), and the soil, and by passive earth pressure on the below-grade walls of the NI Common Basemat Structure Seismic Category I structures; shear keys are not used. Section 2.5.4.2 describes the friction coefficient properties of soil addressed for the U.S. EPR.

Buildings adjacent to the NI Common Basemat Structure are separated from the NI Common Basemat Structure foundation basemat to allow for differential seismic movements between buildings. Refer to Figure 3B-1, which illustrates the gaps between buildings.

Waterproofing membranes used under or within the NI Common Basemat Structure foundation basemat will be evaluated on a site-specific basis, as described in Section 3.8.5.6.

3.8.5.1.2 Emergency Power Generating Buildings Foundation Basemats

Each EPGB foundation basemat supports a building superstructure and associated equipment. At the super-structure and foundation basemat interface, heavily reinforced concrete shear walls function as bearing walls to transfer loads from floors and the roof. Each foundation basemat is embedded approximately five feet into the supporting soil and has overall dimensions of approximately 178 feet long by 94.5 feet wide by 6 feet thick. In the areas of the two diesel fuel oil storage tanks, the foundation basemat reduces in width from 94.5 feet to 42 feet.

Figure 3.8-89 illustrates the general arrangement plan, which also shows the primary shear walls at column lines A, C, E, G and J in the east-west direction, and column lines 11, 13, 17 and 19 in the north-south direction. Additional figures, provided in Appendix 3E, illustrate both the shear walls at the super-structure and foundation basemat interface and the foundation basemat reinforcement.

Figures 3.8-93 and 3.8-94 provide section views of the EPGB structure, which further clarify the relationship between the superstructure and the foundation basemat. Isometric views of the GT STRUDL model representing the overall structure are provided in Section 3.7.2.

3.8.5.1.3 Essential Service Water Buildings Foundation Basemats

The reinforced concrete foundation basemat for each ESWB supports the superstructure and water basin. At the super-structure and foundation basemat interface, heavily reinforced concrete shear walls function as bearing walls to transfer loads from the floors and the roof. Each foundation basemat is embedded approximately 22 feet into the supporting soil and has overall dimensions of approximately 164 feet by ~~128~~108 feet wide by 6 feet thick.

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Figure 3.8-95—Essential Service Water Building Plan Elevation 0'-0"

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Figure 3.8-101—Essential Service Water Building Section A-A

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Figure 3.8-102—Essential Service Water Building Section B-B

Figure 3B-69—Essential Service Water Building Dimensional Plan Elevation 0 m (0 ft)

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Figure 3B-75—Essential Service Water Building Dimensional Section A-A

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Figure 3B-76—Essential Service Water Building Dimensional Section B-B

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3E.3 Essential Service Water Buildings

Description of the Essential Service Water Building Analysis and Design

Four Essential Service Water Buildings (ESWB) are located adjacent to the NI Common Basemat Structures and in the general vicinity of the Emergency Power Generating Buildings (EPGB).

Cross sections and plans associated with each typical ESWB are provided in Section 3.8.4, Figure 3.8-95, Figure 3.8-96, Figure 3.8-97, Figure 3.8-98, Figure 3.8-99, Figure 3.8-100, Figure 3.8-101, and Figure 3.8-102. A general description of the structure, including descriptions of functional equipment at all floor levels, is provided in Section 3.8.4.1.5. ~~It is acknowledged that the GT STRUDL FEM provided in Figure 3.7.2-59 and Figure 3.7.2-60 differs from what is illustrated in this appendix, in that the basemat has been extended 10 ft – 0 in on all sides of the structure. This is further addressed in Section 3E.3.1~~.

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The lateral load resisting system primarily consists of interior and exterior reinforced concrete shear walls and a concrete basemat foundation situated at 22 ft – 0 in below grade. The structural elements pertaining to the ESWBs are described in Sections 3.8.4.1.5 and 3.8.5.1.3.

Upon evaluation of the ESWBs, the following critical sections have been identified:

- Basemat Foundation at Elevation -16 ft – 0 in (3E.3.1).
- Shear Wall at Column Line 4 (3E.3.2).
- Fan Deck Slab at Elevation 63 ft – 0 in (3E.3.3).

Design Criteria

SSI analysis by the Bechtel Code SASSI 2000 (v. 3.1) is used to determine enveloping structural response accelerations for development of equivalent static SSE loads for the GT STRUDL FEM.

The use of GT STRUDL for the design of the critical sections is described in Sections 3.8.4.4.4 and 3.8.5.4.4. Design forces and moments are extracted from GT STRUDL analyses for basemat foundation and superstructure component design.

All applicable loads used for the design of the critical sections located within the ESWBs are described in Sections 3.8.4.3.1 and 3.8.5.3; the applicable loading combinations are described in Sections 3.8.4.3.2 and 3.8.5.3. The design also accommodates the soil analysis cases shown in Table 3.7.1-6.

Reinforced concrete components are designed in accordance with the applicable codes, standards, and specifications described in Sections 3.8.4.2and 3.8.5.2.

The planar reference system for the GT STRUDL finite element analysis output is provided in Figure 3E.3-1—GT STRUDL Finite Element Planar Reference Frame System. The positive direction of the finite element bending moments M_{xx}, M_{yy} and M_{xy} and out-of-plane shear forces V_{xx} and V_{yy} are shown in a) Plate Bending, included on Figure 3E.3-1. The positive direction of the finite element in-plane forces N_{xx}, N_{yy} and N_{xy} are the same as the positive orientation of the plane stresses S_{xx}, S_{yy} and S_{xy} shown in b) Plane Stress/Strain, included on Figure 3E.3-1.

3E.3.1 Basemat Foundation at Elevation -16 ft – 0 in (Top of Concrete)

This critical section presents the structural design of the reinforced concrete basemat required to support the ESWBs. The ESWBs are composed of a 6 ft – 0 in thick reinforced concrete basemat foundation. The basemat foundation of the ESWB is a safety-related, Seismic Category I structure, as described in Section 3.8.5.

Description of the Critical Section and Computer Model

The overall layout and dimensions of each ESWB basemat foundation are described in Section 3.8.5.1.3. ESWBs 1, 2, 3, and 4 are essentially identical; therefore, only one of the ESWBs is evaluated as a critical section.

As described in Section 3.8.5.1.3, GT STRUDL is used to create a FEM to analyze the ESWBs for the forces and moments applied to the ESWB basemat foundation. Figure 3E.3-2—ESWB Basemat Foundation - FEM shows a GT STRUDL FEM view of the ESWB basemat foundation. The typical element size for the elements shown in Figure 3E.3-2 ranges between approximately 4 ft and 7 ft with the aspect ratio kept between 1 and 3.

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The reinforcement sketch for ESWB basemat foundation is provided as ~~As shown in~~ Figure 3E.3-3—Reinforcement Sketch for ESWB Basemat Foundation~~,~~. ~~the basemat foundation extends an additional 10 ft – 0 inches in width on all sides of the structure.~~

Applicable Loadings, Analysis, and Design Methods

In addition to the loads described in Section 3.8.5.3, the GT STRUDL finite element analysis for the basemat foundation for the ESWB incorporates:

- Buoyant forces associated with the high water level (Elevation -1 ft – 0 in, or 21 ft – 0 in of hydrostatic head) for stability design.
- Finite elements representing the superstructure, for accurate load transfer to the basemat foundation.

~~To provide resistance against overturning, the basemat foundation of the ESWB is extended 10 ft – 0 in beyond the building perimeter. The preliminary basemat design is based on boundary conditions defined such that the basemat does not exhibit uplift, which is consistent with the current assumption in Bechtel Code SASSI 2000 regarding soil separation.~~

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Results of Critical Section Design

The structural design for the critical section provides reinforcement to resist element forces and moments as described below.

Table 3E.3-1—Governing Forces and Moments for the ESWB Basemat Foundation shows the governing forces and moments for the design of the ESWB basemat foundation. The section cuts locations are shown in Figure 3E.3-2—ESWB Basemat Foundation–FEM. The sign convention describing the nomenclature for horizontal and vertical cuts applicable to this critical section is shown on Figure 3E.3-10.

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Based on the governing values shown in Table 3E.3-1, the basemat foundation typical reinforcement configuration is shown in Figure 3E.3-3.

Section thicknesses and reinforcing quantities may be optimized based on subsequent analysis results.

3E.3.2 Shear Wall at Column Line 4

This critical section presents the structural design of the reinforced concrete shear wall at column line 4. The typical wall along column line 4 in the ESWB is a safety-related, Seismic Category I structure, as described in Section 3.8.4.

Description of the Critical Section and Computer Model

The reinforced concrete shear wall at column line 4 is selected as a critical section because the wall spans from the basemat foundation elevation at -16 ft – 0 in to the roof elevation at 96 ft - 0 in and has two large openings. The shear wall at column line 4 is shown in Figure 3E.3-7—Reinforcement Configuration for ESWB Wall at Column Line 4.

As described in Section 3.8.3.4.4, GT STRUDL is used to create a FEM to analyze the ESWBs for all forces and moments applied to the ESWB wall at column line 4. The mesh of GT STRUDL elements is established at dimensions ranging approximately between 4 ft and 7 ft with the aspect ratio kept between 1 and 3. A FEM view of the reinforced concrete shear wall at column line 4 is shown in Figure 3E.3-4—ESWB Wall at Column Line 4 - FEM.

Table 3E.3-1—Governing Forces and Moments for the ESWB Basemat Foundation
Sheet 1 of 2

~~Loading Combination~~	~~Joint~~	~~N_{XX} (k/ft)~~	~~N_{YY} (k/ft)~~	~~N_{XY} (k/ft)~~	~~M_{XX} (k-ft/ft)~~	~~M_{YY} (k-ft/ft)~~	~~M_{XY} (k-ft/ft)~~	~~V_{XX} (k/ft)~~	~~V_{YY} (k/ft)~~
~~D + F + L + H + EX + 0.4EY - 0.4EZ~~	~~394~~	**~~-79~~**	~~16~~	~~14~~	~~-82~~	~~-512~~	~~44~~	~~48~~	~~-13~~
~~D + F + L + H + 0.4EX - 0.4EY - EZ~~	~~83~~	~~24~~	**~~-78~~**	~~-19~~	~~-36~~	~~318~~	~~100~~	~~0~~	~~-88~~
~~D + F + L + H + 0.4EX - 0.4EY + EZ~~	~~319~~	~~8~~	~~-17~~	**~~59~~**	~~-85~~	~~-64~~	~~65~~	~~6~~	~~0~~
~~D + F + L + H + EX - 0.4EY + 0.4EZ~~	~~1~~	~~16~~	~~-5~~	~~-0~~	**~~1070~~**	~~872~~	~~-300~~	~~59~~	~~-64~~
~~D + F + L + H + 0.4EX - 0.4EY - EZ~~	~~85~~	~~-13~~	~~27~~	~~-0~~	~~716~~	**~~1655~~**	~~178~~	~~75~~	~~35~~
~~D + F + L + H + EX - 0.4EY - 0.4EZ~~	~~14052~~	~~6~~	~~-20~~	~~-4~~	~~-8~~	~~290~~	**~~374~~**	~~57~~	~~20~~
~~D + F + L + H + EX - 0.4EY - 0.4EZ~~	~~14050~~	~~41~~	~~-23~~	~~10~~	~~191~~	~~408~~	~~208~~	**~~169~~**	~~54~~
~~D + F + L + H + 0.4EX - 0.4EY - EZ~~	~~13931~~	~~-21~~	~~61~~	~~3~~	~~295~~	~~638~~	~~77~~	~~10~~	**~~196~~**
~~D + F + L + H + EX + 0.4EY - 0.4EZ~~	~~394~~	**~~-90~~**	~~17~~	~~15~~	~~-103~~	~~-525~~	~~40~~	~~54~~	~~-10~~
~~D + F + L + H - EX + 0.4EY + 0.4EZ~~	~~14064~~	~~0~~	**~~-80~~**	~~-2~~	~~-2~~	~~59~~	~~-11~~	~~-17~~	~~0~~
~~D + F + L + H + 0.4EX - 0.4EY + EZ~~	~~319~~	~~4~~	~~-19~~	**~~71~~**	~~-76~~	~~-66~~	~~56~~	~~3~~	~~0~~
~~D + F + L + H + 0.4EX + EY - 0.4EZ~~	~~302~~	~~5~~	~~48~~	~~12~~	**~~-738~~**	~~-189~~	~~20~~	~~6~~	~~-9~~
~~D + F + L + H + 0.4EX + 0.4EY - EZ~~	~~523~~	~~13~~	~~-42~~	~~6~~	~~-725~~	**~~-778~~**	~~-68~~	~~57~~	~~54~~
~~D + F + L + H + 0.4EX + EY - 0.4EZ~~	~~508~~	~~-4~~	~~25~~	~~33~~	~~-129~~	~~-48~~	**~~-159~~**	~~-36~~	~~-21~~
~~D + F + L + H + 0.4EX + 0.4EY - EZ~~	~~14179~~	~~-12~~	~~2~~	~~-20~~	~~-54~~	~~-392~~	~~-27~~	**~~102~~**	~~6~~
~~D + F + L + H + 0.4EX + 0.4EY - EZ~~	~~13928~~	~~12~~	~~-50~~	~~1~~	~~-369~~	~~-89~~	~~-7~~	~~6~~	**~~104~~**
~~D + F + L + H + EX + 0.4EY - 0.4EZ~~	~~394~~	**~~-99~~**	~~17~~	~~15~~	~~-122~~	~~-524~~	~~35~~	~~59~~	~~-6~~
~~D + F + L + H - EX + 0.4EY + 0.4EZ~~	~~14064~~	~~-0~~	**~~-88~~**	~~-2~~	~~-2~~	~~63~~	~~-14~~	~~-18~~	~~0~~
~~D + F + L + H + 0.4EX - 0.4EY + EZ~~	~~319~~	~~2~~	~~-22~~	**~~84~~**	~~-65~~	~~-50~~	~~47~~	~~1~~	~~0~~
~~D + F + L + H + 0.4EX + 0.4EY - EZ~~	~~523~~	~~15~~	~~-51~~	~~6~~	**~~-797~~**	~~-832~~	~~-70~~	~~62~~	~~61~~
~~D + F + L + H + 0.4EX + 0.4EY - EZ~~	~~523~~	~~15~~	~~-51~~	~~6~~	~~-797~~	**~~-832~~**	~~-70~~	~~62~~	~~61~~

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Table 3E.3-1—Governing Forces and Moments for the ESWB Basemat Foundation
~~Sheet 2 of 2~~

~~Loading Combination~~	~~Joint~~	~~N_{XX} (k/ft)~~	~~N_{YY} (k/ft)~~	~~N_{XY} (k/ft)~~	~~M_{XX} (k-ft/ft)~~	~~M_{YY} (k-ft/ft)~~	~~M_{XY} (k-ft/ft)~~	~~V_{XX} (k/ft)~~	~~V_{YY} (k/ft)~~
~~D + F + L + H – 0.4EX + 0.4EY – EZ~~	~~14112~~	~~-2~~	~~40~~	~~20~~	~~-13~~	~~-197~~	~~**159**~~	~~-18~~	~~-22~~
~~D + F + L + H + 0.4EX + 0.4EY – EZ~~	~~14179~~	~~-12~~	~~-1~~	~~-23~~	~~-60~~	~~-413~~	~~-19~~	~~**111**~~	~~6~~
~~D + F + L + H + 0.4EX + 0.4EY – EZ~~	~~13928~~	~~13~~	~~-57~~	~~2~~	~~-402~~	~~-87~~	~~-4~~	~~5~~	~~**113**~~

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	Area	Section Cut	Load Combination	FX (kip)	FY (kip)	FZ (kip)	MX (k-ft)	MY (k-ft)	MZ (k-ft)
Out-of-Plane Shear	A1	X6-1	D+L+F (high water) + H(saturated)+EX+0.4EY+0.4EZ	248.6	341.8	186.0	-328.0	NA	43.6
	A2	X1-11	D+L+F (high water) + H(saturated)-0.4EX -0.4EY-0.4EZ	-256.3	-1001.1	-361.0	-5189.0	NA	474.2
	A3	X6-11	1.4D+1.7L+1.4F(high water) + 1.7H(Dry)	0.0	1053.3	0.0	-6481.6	NA	905.6
	A1	Z1-1	D+L+F (high water) + H(saturated)+EX+0.4EY+0.4EZ	370.0	841.5	84.7	20.4	NA	-671.3
	A2	Z13-2	D+L+F (high water) + H(saturated)+EX+0.4EY+EZ	644.2	272.9	362.1	58.3	NA	263.2
	A3	Z25-1	1.4D+1.7L+1.4F(high water) + 1.7H(saturated)	0.0	-1029.3	0.0	1140.9	NA	-3209.4

	Area	Section Cut	Load Combination	FX (kip)	FY (kip)	FZ (kip)	MX (k-ft)	MY (k-ft)	MZ (k-ft)
Compression	A1	X1-1	D+L+F (high water) + H(saturated)-EX -0.4EY-0.4EZ	-102.9	-23.3	-406.9	103.2	NA	-34.4
	A2	X1-12	D+L+F (high water) + H(saturated)-EX -0.4EY-0.4EZ	-263.8	-661.4	-496.4	-2930.6	NA	1039.2
	A3	X6-12	D+L+F (high water) + H(saturated)-EX -0.4EY-0.4EZ	-320.3	146.6	-464.4	-3284.1	NA	1032.5
	A1	Z7-3	D+L+F (high water) + H(saturated)-0.4EX -0.4EY-EZ	-434.7	-520.6	-459.0	-401.5	NA	-2947.3
	A2	Z13-2	D+L+F (high water) + H(saturated)-0.4EX -0.4EY-EZ	-644.2	-273.0	-362.1	-179.8	NA	-1652.4
	A3	Z18-3	D+L+F (high water) + H(saturated)-EX -0.4EY-0.4EZ	-420.9	55.7	-412.6	-835.4	NA	577.5
Tension	A1	X1-1	D+L+F (high water) + H(saturated)+EX+0.4EY+0.4EZ	102.9	-1.5	406.9	502.6	NA	39.8
	A2	X1-12	D+L+F (high water) + H(saturated)+EX+0.4EY+0.4EZ	263.8	-32.1	496.4	-1297.6	NA	1559.0
	A3	X6-12	D+L+F (high water) + H(saturated)+EX+0.4EY+0.4EZ	320.3	598.3	464.4	-1751.4	NA	1657.8
	A1	Z7-3	D+L+F (high water) + H(saturated)+EX+0.4EY+EZ	434.7	-98.0	459.0	52.5	NA	-1749.8
	A2	Z13-2	D+L+F (high water) + H(saturated)+EX+0.4EY+EZ	644.2	272.9	362.1	58.3	NA	263.2
	A3	Z18-3	D+L+F (high water) + H(saturated)+EX+0.4EY+0.4EZ	420.9	338.1	412.6	-494.6	NA	1739.1

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	Area	Section Cut	Load Combination	FX (kip)	FY (kip)	FZ (kip)	MX (k-ft)	MY (k-ft)	MZ (k-ft)
Out-of-Plane Moment	A1	X1-3	1.4D+1.7L+1.4F(high water) + 1.7H(dry)	0.0	226.8	0.0	1534.5	NA	19.1
	A2	X1-11	1.4D+1.7L+1.4F(high water) + 1.7H(dry)	0.0	-866.2	0.0	-5713.4	NA	830.9
	A3	X6-11	1.4D+1.7L+1.4F(high water) + 1.7H(dry)	0.0	1053.3	0.0	-6481.6	NA	905.6
	A1	Z7-1	1.4D+1.7L+1.4F(high water) + 1.7H(saturated)	0.0	-594.4	0.0	-425.9	NA	-5275.1
	A2	Z13-1	D+L+F (high water) + H(saturated) -0.4EX-0.4EY-0.4EZ	-238.4	-189.5	-477.4	-232.2	NA	-2078.2
	A3	Z13-1	1.4D+1.7L+1.4F(high water) + 1.7H(saturated)	0.0	41.1	0.0	-164.3	NA	-9531.0

F (high water): - Loading due to high water level (total water depth = 26 ft).
H (dry): - Loading due to dry soil (dry soil unit weight = 110 pcf).
H (saturated): - Loading due to saturated soil (saturated soil weight = 134 pcf).

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Figure 3E.3-2—ESWB Basemat Foundation - FEM

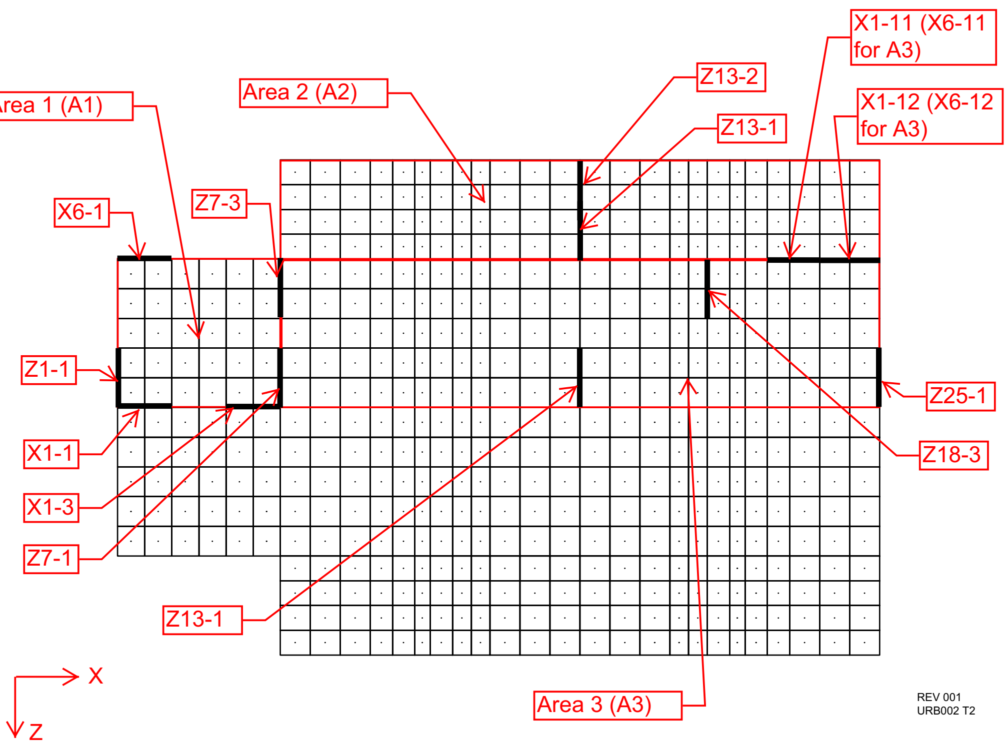

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Figure 3E.3-3—Reinforcement Sketch for ESWB Basemat Foundation

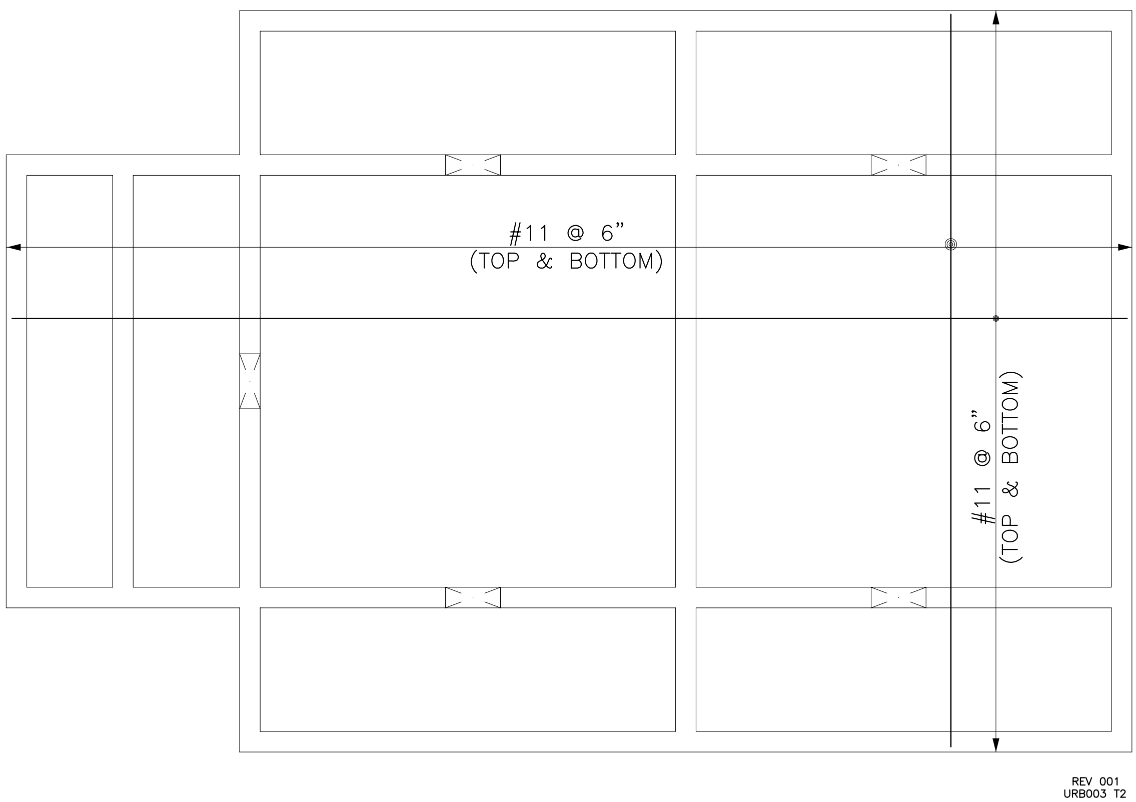

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Figure 3E.3-7—Reinforcement Configuration for ESWB Wall at Column Line 4

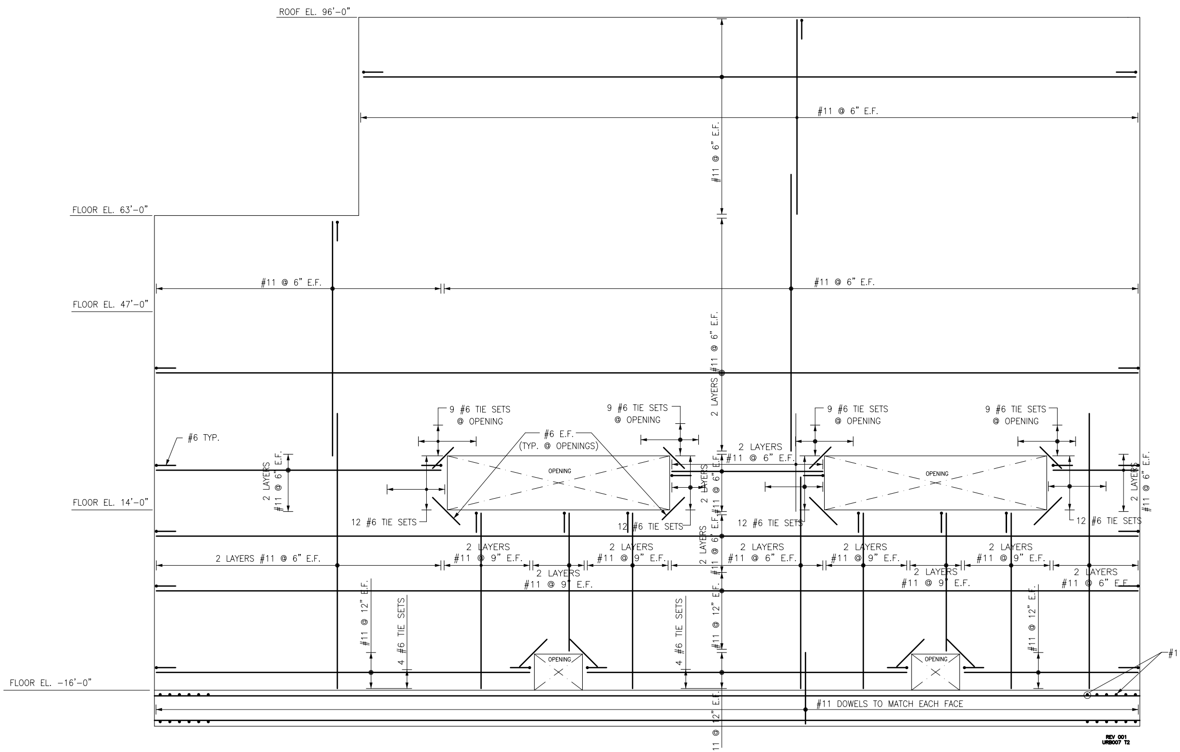

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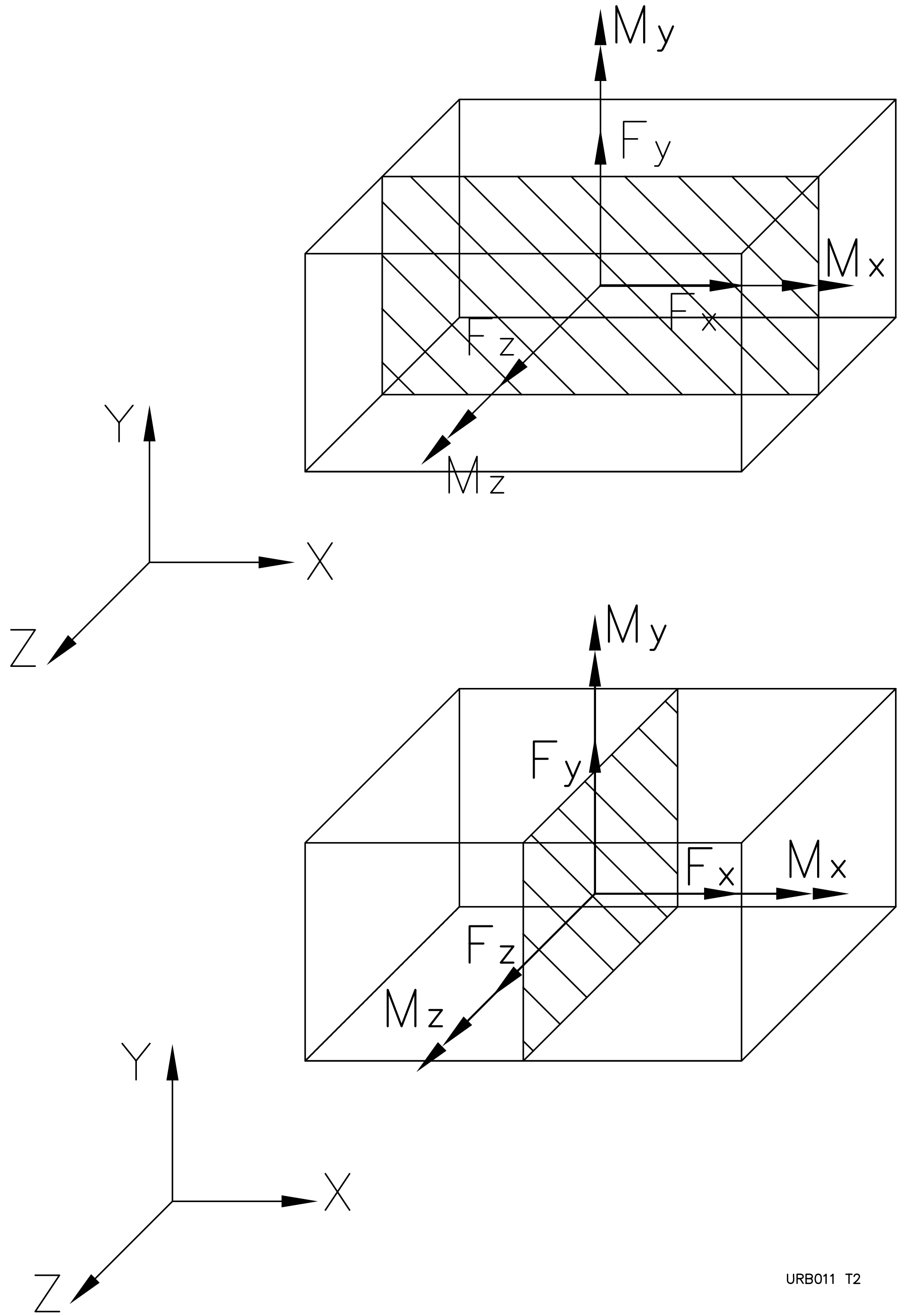
Figure 3E.3-10—Orientation of Positive Axis



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