ArevaEPRDCPEm Resource

From:	WELLS Russell D (AREVA NP INC) [Russell.Wells@areva.com]
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
Attachments:	RAI 130 Supplement 1 Response US EPR DC.pdf

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
RAI 130 — 03.07.01-2	3	3
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RAI 130 — 03.07.01-14	9	9
RAI 130 — 03.07.01-15	10	11
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RAI 130 — 03.07.01-18	13	13
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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)
Ronda Pederson
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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
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Hearing Identifier: AREVA_EPR_DC_RAIs
Email Number: 244

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Subject: Response to U.S. EPR Design Certification Application RAI No. 130, FSAR Ch 3, Supplement 1
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MESSAGE	7986	2/20/2009 7:30:35 PM
RAI 130 Supplement 1 Response US EPR DC.pdf		290740

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Response to

Request for Additional Information No. 130, Supplement 1 (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 03.07.01-1:

In FSAR Section 3.7.1.1.1 it states that for seismic analysis the point of seismic input for the CSDRS is an outcrop or hypothetical outcrop at the foundation elevation of the Nuclear Island basemat which is at -12.60 meters (-41.33 ft) below the surface of the ground. The determination of seismic input for other Seismic Category I structures is determined from the CSDRS modified to account for structure-soil-structure interaction (SSSI) between the NI common basemat structures and the Emergency Power Generating Buildings (EPGBs) and the Essential Service Water Buildings (ESWBs). In this SSSI analysis, described in FSAR Section 3.7.2.4.4, the foundations of both the EPGB and the ESWB are taken to be at the same elevation as the NI common basemat foundation as shown FSAR Figure 3.7.2-63. The actual elevation for the EPGB is at grade and the ESWB is embedded at -6.70 m (-22 ft) below grade. Provide the basis for not accounting for the differences in elevations of these structures and address the impact on the development of the modified CSDRS.

Response to Question 03.07.01-1:

The effects of the soil above the NI basemat elevation are addressed by the COL applicant, as required by COL Information Item 2.5-3 and described in U.S. EPR FSAR Section 2.5.2.6. The foundation input response spectra (FIRS) for the EPGB and ESWB are determined during COLA, taking into consideration the soil up to the ground surface.

FSAR Impact:

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

Question 03.07.01-2:

In Section 3.7.1, the proposed U.S. EPR certified seismic design response spectra (CSDRS) (in two horizontal and one vertical direction) consist of three individual design response spectra for three EUR control motions corresponding to hard, medium, and soft sites. However a COL applicant will need to compare site specific ground motion response spectra (GRMS) with one set of the EUR control motions depending on whether the specific site is a hard, medium, or soft soil site. As such, include in Tier 1 Section 5.0 separate figures depicting the three individual CSDRS for 5% damping (including the design peak ground acceleration level and the spectral shape) corresponding to hard, medium, and soft sites.

Response to Question 03.07.01-2:

The European Utility Requirements (EUR) control motions were added to U.S. EPR FSAR Tier 1, Figure 5.0-1 in the Response to RAI 35, Question 02.05.02-4.

FSAR Impact:

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

Question 03.07.01-3:

In FSAR 3.7.1, the U.S. EPR CSDRS is not based on a single control motion which envelopes all the individual design response spectra (DRS) for hard, medium, and soft sites. As such, a COL applicant has to make a determination as to which category (i.e., hard, medium, or soft) the local site falls into and then to meet the acceptance criteria of the SRP and verify that site specific GMRS is enveloped by the corresponding CSDRS. Provide specific criteria in a COL information item for categorizing a site as hard, medium, or soft.

Response to Question 03.07.01-3:

This question was addressed in the Response to RAI 35, Question 02.05.02-2.

FSAR Impact:

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

Question 03.07.01-4:

In FSAR 3.7.1, Standard Plant Generic soil parameters (i.e., shear wave velocity and damping values) used for the SSI analysis are considered to be the final strain compatible values. Thus, a COL applicant needs to compare its final iterated site soil parameters (when calculating GMRS) with the standard plant generic soil parameters when categorizing the site as hard, medium, or soft. Provide this clarification in the appropriate COL information item.

Response to Question 03.07.01-4:

The COL applicant is required to compare the final iterated soil parameters with the standard plant generic soil parameters, as required by COL Information Item 2.5-3. This COL item is further clarified in U.S. EPR FSAR Section 2.5.4.7 and 2.5.2.6, Step 5. In the SSI analysis, the soil parameters are assumed to be strain-compatible.

FSAR Impact:

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

Question 03.07.01-9:

On FSAR Page 3.7-9 in the last paragraph, it is stated that the characteristics (V/A & AD/V^2) of synthetic time histories are generally consistent with the characteristic values for the magnitude and distance of "the appropriate controlling" events defined for the UHRS. These characteristics are one of the review requirements of the SRP. Therefore, provide further clarification with regard to the magnitude and distance of the controlling event. Alternatively, this should be a COL information item requiring a COL applicant to respond to as part of the site specific analysis.

Response to Question 03.07.01-9:

For generic site conditions in certified standard plants, the properties are assumed values (see U.S. EPR FSAR Tier 2, Section 3.7.1.1.1). These assumed properties will be reconciled by the COL applicant as required by COL Information Items 2.5-3 and 3.7-1 and further described in U.S. EPR FSAR Section 2.5.2.6 and Section 3.7.2, respectively. Magnitude and distance of the controlling event for the site specific ground motion calculation will be determined during development of site specific ground motion response spectra (GMRS). Before reconciliation of site specific ground motion can be calculated, the controlling event must be identified and its characteristics defined. Thus, no additional COL Information Items are required.

FSAR Impact:

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

Question 03.07.01-12:

In FSAR Section 3.7.1.2 (First Paragraph on Page 3.7-11), it is stated that the ISRS generated for the Emergency Power Generating Buildings and the Essential Service Water Buildings are based on OBE structural damping. The staff agrees with this approach but would like to include in an appropriate Table (as part of the FSAR) of the specific OBE level structural damping values used for the analysis. Provide such a table.

Response to Question 03.07.01-12:

U.S. EPR FSAR Tier 2, Table 3.7.1.1, Note 4 will be revised:

> "A damping value of four percent has been used for generation of the ISRS for the EPGB and ESWB."

Use of the operating basis earthquake (OBE) damping value to generate ISRS pertaining to Emergency Power Generating Buildings (EPGB) and Essential Service Water Buildings (ESWB) is provided in U.S. EPR FSAR Tier 2, Table 3.7.1-1.

FSAR Impact:

U.S. EPR FSAR Tier 2, Table 3.7.1.1 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.01-13:

FSAR Table 3.7.1-1 lists damping values for the reactor coolant system. A damping value of 7 percent is listed for the RPV closure head equipment tie rods. Verify that the tie rod connection represents a bearing connection as opposed to a friction connection.

Response to Question 03.07.01-13:

Reactor pressure vessel (RPV) closure head equipment tie rod ends are connected to supporting and supported elements with pins and clevises using bearing connections. Therefore, an applied damping value of seven percent is appropriate.

FSAR Impact:

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

Question 03.07.01-14:

In FSAR Section 3.7.1.2 (Last Paragraph on Page 3.7-10), it indicates that the Rayleigh mass and stiffness weighted damping coefficients for the reactor coolant systems are selected to provide generally conservative damping across the frequency range of interest relative to the values in Table 3.7.1-1. Specify the frequency range of interest for the calculations to assure that the frequency range is sufficient.

Response to Question 03.07.01-14:

U.S. EPR FSAR Tier 2, Section 3.7.1.2 addresses damping used in the non-linear RPV isolated model seismic analysis. The direct step-by-step integration time history solution technique with Rayleigh damping was used for the RPV isolated model, which is non-linear due to the various gaps within the RPV internals. Such an approach automatically incorporates natural frequencies of the system without requiring a modal analysis to determine or to restrict the frequencies to a certain range. Therefore, the frequency range of interest for such a solution technique is by default, from zero to infinity.

FSAR Impact:

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

Question 03.07.01-15:

FSAR Table 3.7.1-1 specifies the damping value for cable trays with flexible support systems as no more than 20%. While RG 1.61, Rev. 1 lists a 10% damping value for cable tray systems, it permits the use of higher damping values subject to obtaining NRC review for acceptance on a case by case basis. As such, provide the technical basis (including actual test data and studies and their applicability and limitations, etc) of using a damping value of no more than 20% for cable trays with flexible support in the EPR application.

Response to Question 03.07.01-15:

U.S. EPR FSAR Tier 2, Table 3.7.1.1 Notes 3B and 3D will be revised and Note 3E will be added as follows:

“B. Spare and initially empty cable tray(s) may be analyzed with zero cable load and these damping values. (Reanalysis required when system is put into service).”

“D. Selected damping value is to be justified and documented on an individual basis when cable loadings less than 50 percent of the maximum rated loading are specified for design calculations.”

“E. Higher damping values limited to flexibly supported rod and strut-hung trapeze systems, and strut-type cantilever and braced-cantilever cable tray systems loaded to greater than 50 percent of the maximum rated loading.”

U.S. EPR FSAR Tier 2, Table 3.7.1.1 “Damping Values for Safe Shutdown Earthquake” will be revised to include the following:

- “Flexible Support Systems[E]“

U.S. EPR FSAR Tier 2, Appendix 3A.3.5 will be revised to remove the circular logic.

Use of a damping value up to 20 percent for cable tray systems is based on results of a test program performed by ANCO Engineers, Inc., in cooperation with Bechtel Power Corporation (“Cable Tray and Conduit Raceway Seismic Test Program, Release 4,” Report 1053-21.1-4, ANCO Engineers Inc., December 15, 1978). Bechtel maintains the referenced ANCO report as proprietary information that is not publicly available. Bechtel submitted this report to the NRC as proprietary information and it is referenced in NUREG/CR-6919 “Recommendations for Revision of Seismic Damping Values” and in RG 1.61, 2006. NUREG/CR-6919, Section 5.2.1 references the ANCO test program, which identifies heavily loaded flexibly supported rod and strut-hung trapeze systems, and strut-type cantilever and braced-cantilever cable tray systems achieve damping values as high as 20 percent to 25 percent with lower damping values for un-loaded and lightly loaded cable trays. The referenced ANCO report and associated testing is the basis for using damping values up to 20 percent for cable tray systems at the Seabrook Station, Units 1 and 2 (Refer to NUREG-0896, Supplement 5) with NRC approval. Bechtel Power Corporation is a subcontractor to AREVA for assisting with preparation of specific sections of the U.S. EPR FSAR and has identified the referenced ANCO report as available for review in its Frederick, Maryland offices.

ASCE/SEI 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities," American Society of Civil Engineers, 2005 identifies additional support systems that are both similar to and significantly different than those evaluated using the Bechtel/ANCO test program. A damping value of 15 percent is widely recognized by the industry as acceptable for heavily loaded cable tray systems that are less flexibly supported.

FSAR Impact:

U.S. EPR FSAR Tier 2, Table 3.7.1.1 and Appendix 3A.3.5 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.01-16:

The FSAR Section 3.7.1.3 (Last Paragraph on Page 3.7-12) indicates that soil densities in the SSI analysis vary from 1760 to 2000 kg/cubic meter (110 to 125 pcf). However, Table 3.7.2-9 indicates a weight density of 2496 kg/cubic meter (156 pcf) for soil case with a shear wave velocity of 4000 m/sec (13123 ft/sec) (soil case no. 5a). Confirm that the soil weight density of 2496 kg/cubic meter (156 pcf) was used for Soil Case No. 5a in the SSI analysis in Section 3.7 and make appropriate corrections in FSAR page 3.7-12 last paragraph.

Response to Question 03.07.01-16:

The SSI analysis uses a soil weight density of 2496 kg/cubic meter (156 pcf) for Soil Case No. 5a as stated in U.S. EPR FSAR Tier 2, Section 3.7. U.S. EPR FSAR Tier 2, Section 3.7.1.3 will be revised as follows:

> "…SSI analysis varies from 110 to 156 pcf for soil."

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 03.07.01-18:

Appendix S to 10 CFR Part 50 states that the horizontal component of the SSE ground motion occurring at the foundation level in the free field must be an appropriate response spectrum with a peak ground acceleration of at least .1g. For the U.S. EPR standard plant, the required minimum design basis spectra for the NI common basemat structures is provided by an envelope of the three EUR design response spectra modified to reflect a peak ground acceleration of .1g. In FSAR Figure 3.7.1-2, a comparison is made showing the CSDRS bounding the minimum required spectra anchored at .1g. Response spectra meeting the requirements of Appendix S for appropriate response spectra with a peak ground acceleration of at least .1g has not been provided for either the Emergency Power Generating Buildings (EPGBs) or the Essential Service Water Buildings (ESWB). Provide and include in the FSAR a minimum response spectra meeting, and its basis for meeting, the requirements of Appendix S for these safety-related structures.

Response to Question 03.07.01-18:

The minimum response spectra for the EPGB and ESWB are the same as for NI Common Basemat structures shown in U.S. EPR FSAR Tier 2, Figure 3.7.1-2.

FSAR Impact:

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

Question 03.07.02-2:

In FSAR Section 3.7.2.1.3 (pg 3.7-69, 4th paragraph), it indicates that the complex frequency response analysis method is also used in the soil column analysis using Bechtel computer code SHAKE2000, Version 1.1, to compute the free-field "in-ground" motion at the foundation level of ESWBs, for use as the input motion to the SSI analysis. This is indicated to be needed to incorporate the effects of embedment in the SSI analysis of the ESWBs. The input ground motion specified in Section 3.7.1 corresponds to a hypothetical free-field "outcrop" motion at the foundation level of ESWB. Bechtel code SASSI 2000 requires that the input motion, when specified at the foundation level, must be an "in-ground" motion converted from the "outcrop" motion through a soil column analysis. Please indicate if, in generating "outcrop" motions using the SHAKE Code whether the soil column above the foundation level (a depth of about 6.7 m (22 ft)) below grade) is removed from the soil column as required by both the SRP and the ISG. The "outcrop" must be defined assuming no soil above the level of the "outcrop" depth and all potential effects of down-coming waves need to be removed from the computation. Provide analytic modeling, outcrop and in-ground spectra, FIRS, soil conditions and numerical results to indicate how the SHAKE computations are performed.

Response to Question 03.07.02-2:

U.S. EPR FSAR Tier 2, Section 3.7.2.1.3 will be revised as follows:

> "The complex frequency response analysis method is also used in the soil column analysis using Bechtel computer code SHAKE2000, Version 1.1, to compute the freefield, "in-column" motion at the foundation level of ESWB, for use as the input motion to the SSI analysis."

U.S. EPR FSAR Tier 2, Section 3.7.2.1.3 will be revised:

> "Bechtel code SASSI 2000 requires that the input motion, when specified at the foundation level, be an "in-column" motion converted from the "outcrop" motion through a soil column analysis."

U.S. EPR FSAR Tier 2, Section 3.7.2.16, Reference 10 will be added:

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).

SHAKE2000 was used to generate an "in-column" motion that was used as input motion in SASSI2000 for the SSI analysis of ESWB. Analyses of the soil profiles of the generic sites include soil layers between the ground surface and foundation level of ESWB. The input motion is applied as "outcrop" motion at the foundation level of ESWB and the response motion is the "in-column" motion also at the foundation level. The soil column and its respective "in-column" motion are used in the SASSI2000 SSI analysis of ESWB for each of the ten soil profiles.

Embedment of the ESWB is considered and modeled in the soil-structure interaction analyses of ESWB. The inclusion of the soil column above the foundation level of ESWB in the SHAKE2000 analyses is consistent with the SSI analyses that use the "in-column" motion as control motion. The methodology used in the analyses is consistent with the methodology provided in Reference 10.

The input and output data for the soil column analysis using SHAKE2000 may be summarized as follows:

- The generic soil column models are shown by the plot of shear wave and P-wave velocities in Figure 03.07.02-2-1 (same as shear wave velocities shown in U.S. EPR FSAR Tier 2, Figures 3.7.1-31 and 3.7.1-32) and Figure 03.07.02-2-2. No iteration is allowed for these SHAKE analyses.
- The foundation input response spectra (FIRS) for Essential Service Water Building (ESWB) is conservatively assumed to be the enveloped of (1) Soft, Medium, and Hard EUR control motions and (2) the response motions at the foot prints of Emergency Power Generating Building (EPGB) and ESWB from NI soil-structure interaction (SSI) analyses for all 10 soil profiles using the soft, medium, and hard EUR control motions as input. These motions conservatively include the structure-to-structure interaction. The enveloping motion is assumed to be the FIRS for ESWB.
- The "assumed" FIRS for the ESWB are used as outcrop motion for soil column analysis. The time history matching the assumed FIRS and the "in-column" response motion of the 10 soil columns are shown in Figures 03.07.02-2-3, 03.07.02-2-4 and 03.07.02-2-5, respectively for x, y, and z directions in the form of plots of acceleration response spectra.

FSAR Impact:

U.S. EPR FSAR Tier 2, Sections 3.7.2.1.3 and 3.7.2.16 will be revised as described in the response and indicated on the enclosed markup.

Figure 03.07.02-2-1—Shear Wave Velocities of the Generic Soil Profiles

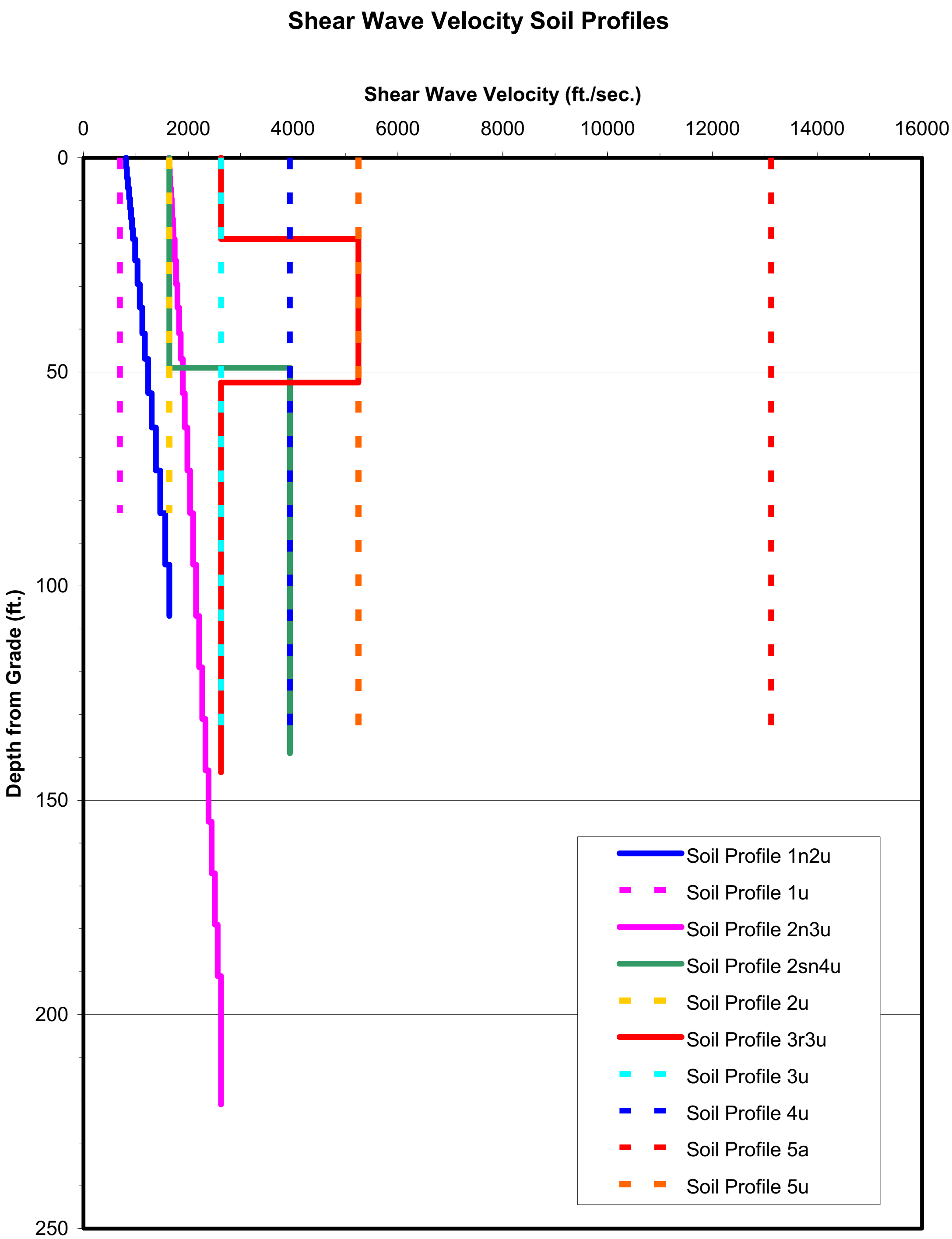


Figure 03.07.02-2-2—P-wave Velocities of the Generic Soil Profiles

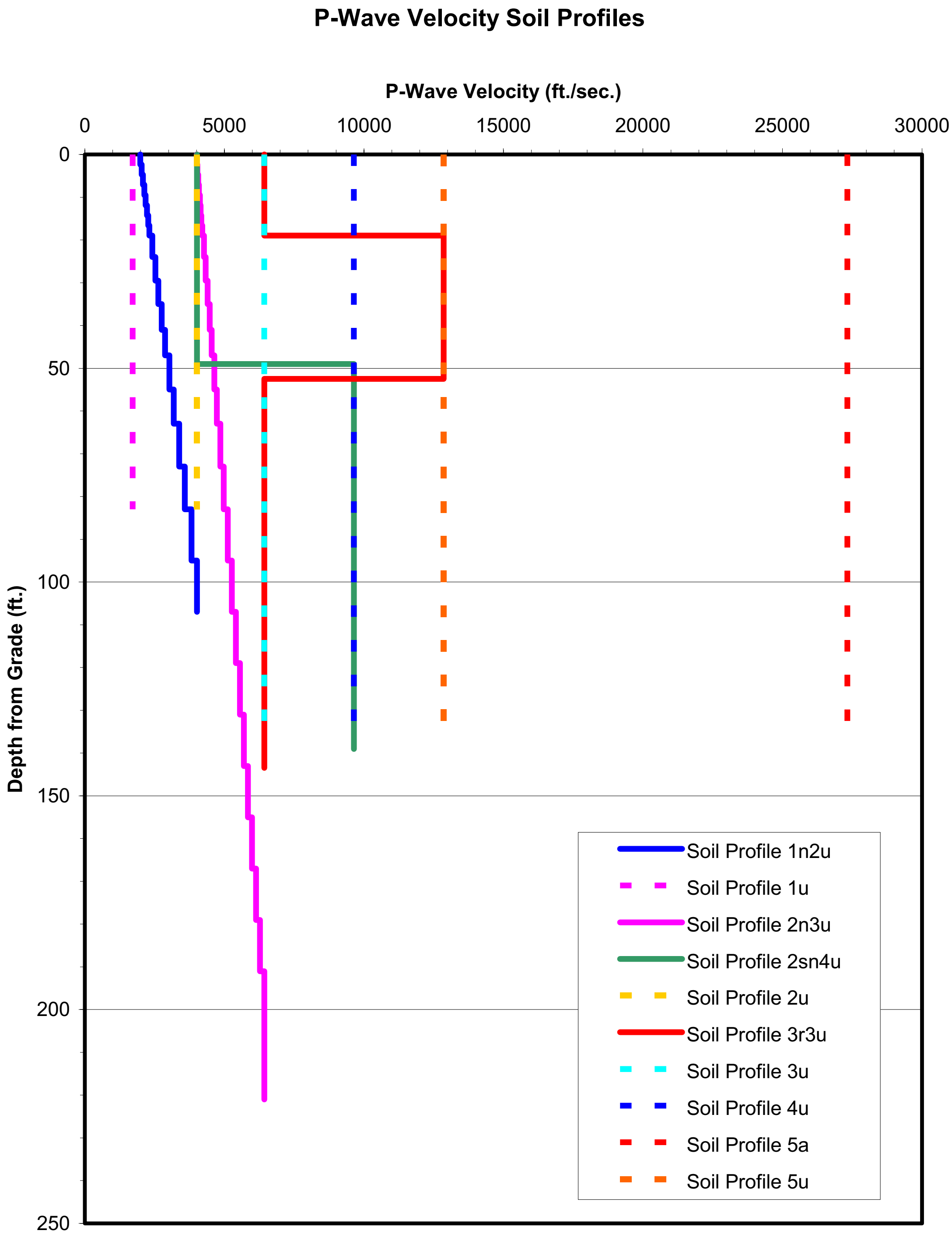


Figure 03.07.02-2-3—Comparison of Ground Motions in the X-direction

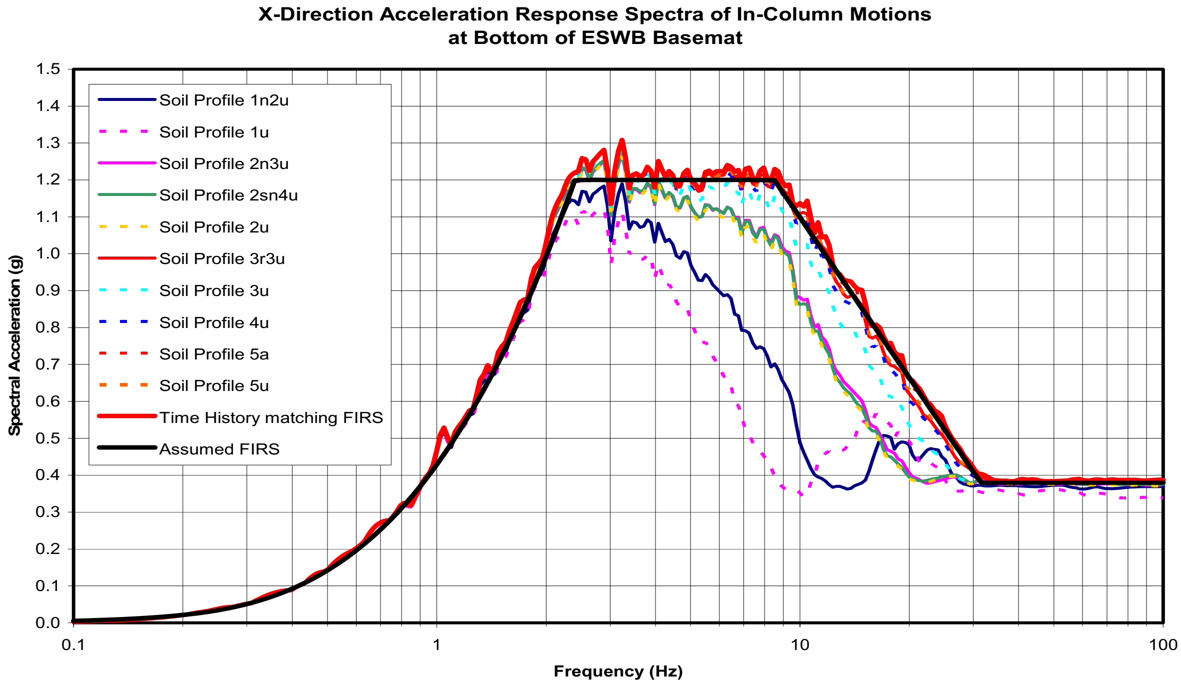


Figure 03.07.02-2-4—Comparison of Ground Motions in the Y-direction

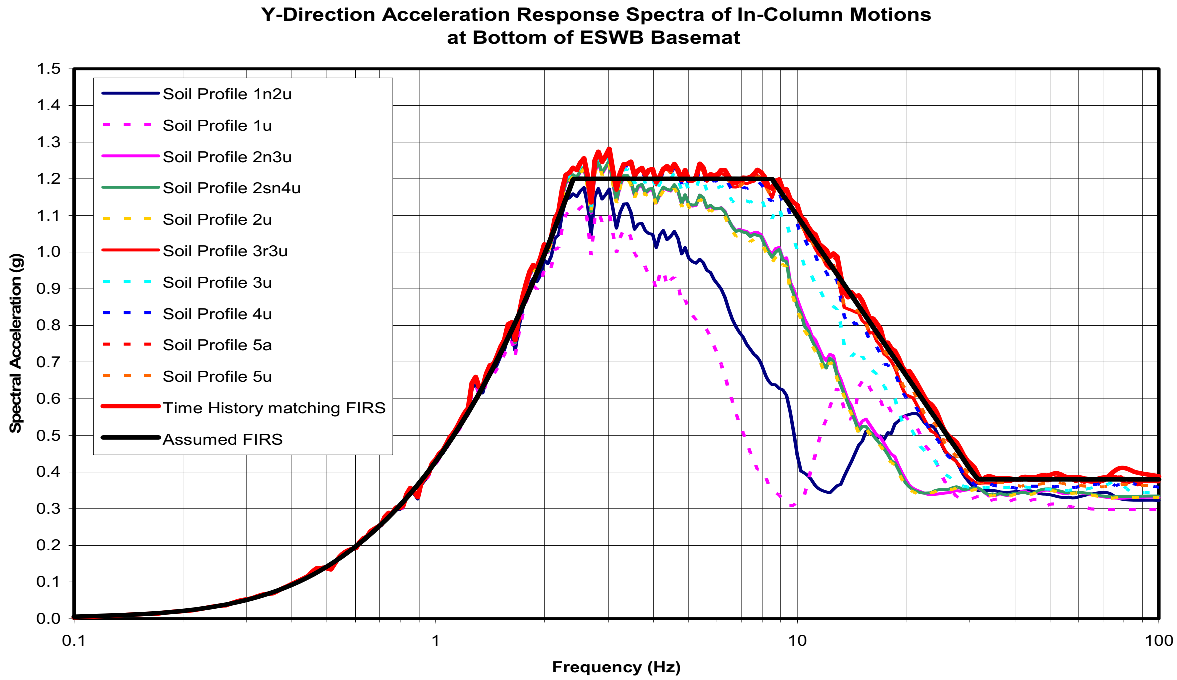
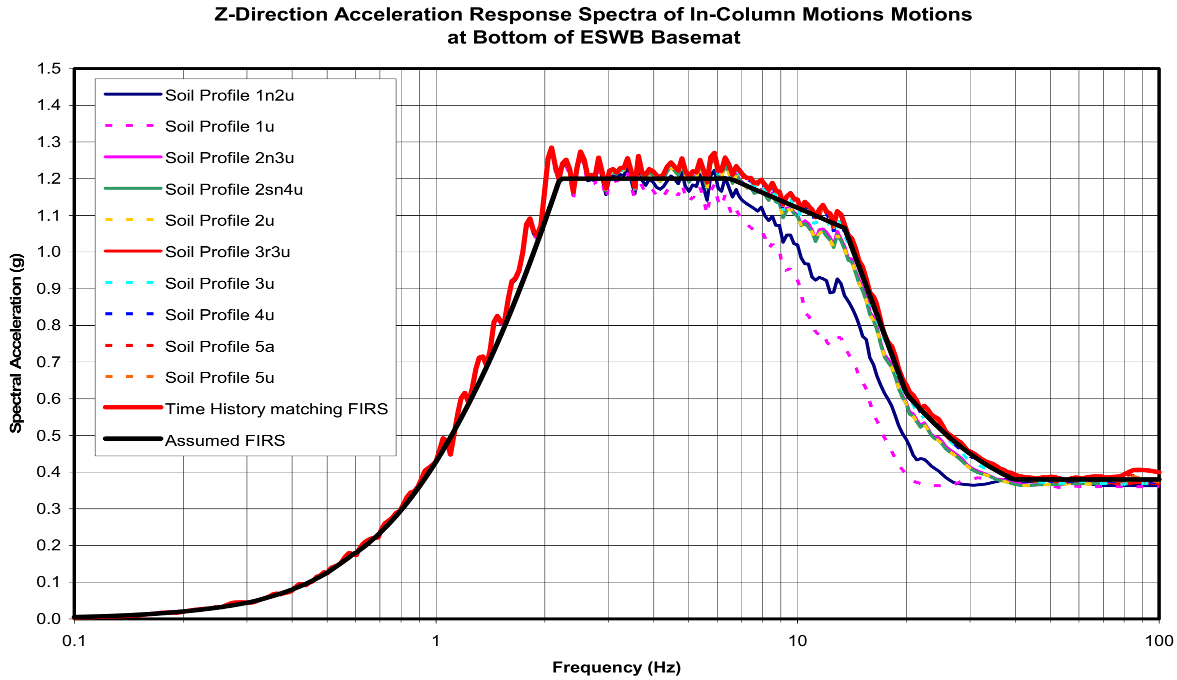


Figure 03.07.02-2-5—Comparison of Ground Motions in the Z-direction



Question 03.07.02-8:

In FSAR Section 3.7.2.1.1 (pg 3.7-67), it states that when nonlinearities occur in the stiffness matrix or damping matrix, the direct integration technique is used. It states that this technique is used for the time history analysis of the NI common basemat structures to determine their stability against seismic sliding or overturning and their potential for seismic structural interaction.

a. Provide the basis and values used for the stiffness and damping matrices.
b. Describe the design motion time histories that are used in the nonlinear structural analysis and the basis for their selection.
c. Describe how the soil springs are modeled in this analysis.
d. Provide the relationship between this analysis and the seismic analysis conducted to determine structural loads and ISRS.

Response to Question 03.07.02-8:

U.S. EPR FSAR Tier 2, Section 3.8.5.4.2 describes the nonlinear model. In particular:

a) The stiffness and damping parameters are based on the SASSI transfer functions. Stiffness values range from $1.09*10^{8}$ – $6.73*10^{10}$ lb/inch while damping values range from $1.17*10^{7}$ – $2.30*10^{8}$ lb-s/inch, depending on the soil case and direction.

b) The design motion time histories used in the nonlinear analysis are the same motions used in the SASSI analyses, namely, the EUR ground motion time histories.

c) Soil springs are modeled to allow sliding and uplifting to occur at the soil-structure interface. Three soil springs and three dampers (one each in the x, y, and z directions) are placed at each node along the bottom of the basemat. The stiffness and damping parameters are distributed across the soil springs and dampers based on the contributing area associated with each node to create an even distribution along the bottom of the basemat. The soil springs are not directly attached to the bottom of the basemat, but instead interact with the superstructure only through contact and target elements to allow sliding and uplifting. These contact and target elements are used to model the frictional forces between the superstructure and the soil.

d) The SASSI analysis is used to determine building accelerations and in-structure response spectra (ISRS). The nonlinear analysis is used to check stability only.

FSAR Impact:

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

Question 03.07.02-10:

In FSAR Section 3.7.2.1.2 (pg 3.7-68), it states that the response spectrum method is used in the NAB for local seismic analysis of certain slabs to determine their out of plane seismic loads. What is the basis for determining which slabs in the NAB use this method of analysis and how is the local analysis performed?

Response to Question 03.07.02-10:

U.S. EPR FSAR Tier 2, Section 3.7.2.1.2 will be revised as follows:

> “Response spectrum analyses are performed on flexible long span floors and roof of the NAB Non-Seismic Category I structure to obtain the amplified vertical accelerations of the floors.”

The Nuclear Auxiliary Building (NAB) Non-Seismic Category I structure is analyzed using time-history and lumped mass stick model analysis methods in lieu of using lollipops to capture the amplified floor response and in-structure response spectra (ISRS). The same is accomplished for representative long-span flexible floors by subsequently conducting response spectrum analysis for the individual floors. This assures that the analyzed floors are not under-designed for vertical seismic loads.

The roof slab of the Non-Seismic Category I NAB is specifically investigated to confirm that the slab would not collapse during a safe shutdown earthquake (SSE) event. The roof slab of the NAB is selected because of its long spans and relative flexibility as compared to other slabs in the structure.

A global dynamic analysis that considers soil-structure interaction (SSI) effects for the structure is performed using a lumped-mass stick model as described in US EPR FSAR Tier 2, Sections 3.7.2.1.1 and 3.7.2.3.1. ISRS are developed at slab locations from the global analysis.

FSAR Impact:

U.S EPR FSAR Tier 2, Section 3.7.2.1.2 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.02-17:

In FSAR Section 3.7.2.4.7 (pg 3.7-90), it states that for the EPGBs and the ESWBs, the 3D FEM of the structures is sufficient to represent the flexible slabs and wall in cracked conditions, while the SDOF oscillators added to the 3D FEM represent the un-cracked condition. Are all walls and floors in the FEM assumed to be in the cracked condition? How are the cracked properties represented in the FEM? How do SDOF oscillators get represented in the model? Normally, a cracked condition and then an un-cracked condition would be analyzed and an envelope of results selected for further analysis and design. How does the modeling described accomplish this? How are the results applied to determine the structural design loads and the development of ISRS?

Response to Question 03.07.02-17:

(a) All walls and elevated slabs used cracked concrete properties in the out-of-plane direction.

(b) Cracked properties are modeled by setting Young's modulus of the cracked concrete equal to 50 percent of the un-cracked modulus.

(c) Single-degree-of-freedom (SDOF) oscillators in the Emergency Power Generating Building (EPGB) finite element model are used to determine the out-of-plane responses. The fundamental modal frequency of individual slabs and walls are calculated. The mass and support stiffness of each oscillator are adjusted so that its natural frequency of vibration equals to the calculated slab/wall frequency. Each oscillator has two supports at opposite edges of the slab or wall. For one-way slabs (or walls), the two support points are located at the support edges of the short span. For two-way slabs, the two support points chosen are at the opposite edges that have the highest responses. It is noted that the mass assigned to each lollipop was kept very small to establish that the global seismic response of the structure remained unaffected.

(d) Both EPGB and Essential Service Water Building (ESWB) are box-shaped shear wall building structures. The overall building stiffness and building seismic responses are dominated by the in-plane stiffness of the shear walls while cracking in the out-of-plane directions, and has only a minor influence. Therefore, the overall responses of EPGB and ESWB are not sensitive to cracking or un-cracking of walls in the out-of-plane directions.

    Because the frequencies associated with un-cracked out-of-plane behavior for wall and slab frequencies are relatively high and away from the predominant frequencies of the seismic motions, it is determined that the responses from the cracked walls and slab will be controlling. This is confirmed by cracked oscillator responses, which are higher than the corresponding un-cracked oscillator responses.

(e) Maximum accelerations at the slab elements and wall elements are used to determine the overall seismic loads and ISRS. Out-of-plane maximum accelerations at the SDOF oscillators are used to determine the local out-of-plane seismic load on individual slabs and walls. Procedures used in the development of seismic loads are addressed in the Response to Question 03.07.02-34.

FSAR Impact:

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

Question 03.07.02-22:

In FSAR Section 3.7.2.14 (pg 3.7-100), it states that the overturning of the common basemat of the NI structures due to a seismic event does not occur due to its inherent stability. In this regard, describe the analytical model for assessing building stability during a seismic event and provide the corresponding factors of safety against potential sliding (including maximum absolute displacement of NI common basemat structure) and overturning during the design basis seismic event. Also, what is the assumed minimum coefficient of friction used for evaluation of the translational stability of the NI structures? Is there a requirement for a COL applicant to meet a minimum coefficient of friction to be available at the soil/basemat interface? If so, specify this information in Table 2.1-1 of the FSAR. Similar information should be provided for the EPGBs and the ESWBs.

Response to Question 03.07.02-22:

The U.S. EPR FSAR requires a COL applicant to meet or exceed a minimum coefficient of friction at the soil/basemat interface. U.S. EPR FSAR Tier 2, Table 1.8-2, U.S. EPR Combined License Information Items, Item 3.8-10, states, “A COL applicant that references the U.S. EPR design certification will evaluate site-specific methods for shear transfer between the foundation basemats and underlying soil for soil parameters that are not within the envelope specified in Section 2.5.4.2.”

U.S. EPR FSAR Tier 2, Section 2.5.4.2 states:

The following soil properties are used for design of U.S. EPR Seismic Category I structures.

- Coefficient of friction acting on foundation basemats and near surface foundations for Seismic Category I structures = 0.7

U.S. EPR FSAR Tier 2, Table 2.1-1 will be revised to reflect this as follows:

Minimum Coefficient of Static Friction	0.7
(representative of soil-basemat interface)	

Overturning and sliding will be specifically addressed in the Response to RAI 155, Question 03.08.05-8 for the Nuclear Island and the Response to RAI 155, Questions 03.08.05-9 and 03.08.05-12 for the EPGB and ESWB.

FSAR Impact:

U.S. EPR FSAR Tier 2, Table 2.1-1 will be revised as described in the response and indicated on the enclosed markup.

Question 03.07.02-24:

In FSAR Section 3.7.2.4.4 (Line 7 from bottom of Page 3.7-86), it is stated that the SSI of the NI common basemat structures will have some effect on EPGBs and ESWBs. It states that this effect has been captured by modeling the surrounding footprints on the soil surface along with the NI common basemat SSI model. Accordingly, confirm that the modified CSDRS used for the analysis of the surrounding category I structures represent the envelope of soil surface response spectra calculated from each of ten generic soil profile SSI analyses (NI common base mat and NAB) at the EPGB and ESWB footprints.

Response to Question 03.07.02-24:

The modified Certified Seismic Design Response Spectra (CSDRS) used for the analysis of the surrounding category I structures is confirmed to represent the envelope of soil surface response spectra, which is calculated for ten generic soil profile soil–structure interaction (SSI) analysis as described in U.S. EPR FSAR Tier 2, Sections 3.7.2.4.4 and 3.7.2.4.5.

FSAR Impact:

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

Question 03.07.02-32:

In the acceptance criteria of SRP 3.7.2-SAC-3.E, it states that the method for transferring the seismic response load from the dynamic model to the structural model used for the detailed design should be reviewed for technical adequacy. What is the process that is used to accomplish the load transfer from the seismic analysis models to the analysis and design models? Include this information in the FSAR.

Response to Question 03.07.02-32:

The maximum zero period accelerations (ZPA) calculated from the soil–structure interaction (SSI) analysis are used to apply seismic loads to the static model. Masses in the static model are accelerated by the ZPA values, while forces and pressures (e.g., representing equipment weights) are scaled by the ZPA values. See U.S. EPR FSAR Tier 2, Section 3.7.2.1.4 and U.S. EPR FSAR Tier 2, Section 3.8.4.4.1.

FSAR Impact:

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

Question 03.07.02-33:

In FSAR Section 3.7.2.3.1 on page 3.7-72, it states that for pools the frequency of the water sloshing is typically low compared to the first horizontal mode frequency of the structure housing the pool. Therefore, the water sloshing has a negligible effect on the response of the structure and can be ignored in the development of the stick model. The effect is considered in the local analysis and detailed design of the pool. The staff would like examples of the sloshing frequency provided and compared with the fundamental mode of the structural frequency. In addition, describe the model and process for taking sloshing into account in the local analysis of the pool.

Response to Question 03.07.02-33:

The sloshing frequency ranges from approximately 0.1 Hz to 0.5 Hz. The fundamental frequency of the associated pool structures ranges from approximately 2.5 Hz to 13 Hz.

Convective forces resulting from the sloshing of water are calculated based on the natural frequency of the sloshing water. When the natural frequency is calculated, the spectral acceleration of the sloshing water is obtained from the 0.5 percent damping curve, as stated in U.S. EPR FSAR Tier 2, Section 3.7.3.14. The convective force is then calculated using the spectral acceleration in accordance with the equations in TID-7024, “Nuclear Reactors and Earthquakes”, 1963. The calculated convective force is applied as a pressure to the tank walls and used for design of the tank structure.

FSAR Impact:

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

Question 03.07.02-34:

In FSAR Section 3.7.2.6(2) (pg 3.7-94) for the EPGBs and ESWBs, it states that the three components of earthquake motion are combined using the (1.0, .4, .4) rule. This meets the requirements of RG 1.92, revision 2 and is acceptable for determining the response of the structure. In FSAR Section 3.7.2.4.6 on page 3.7-89, it states that for each of the ten generic soil cases, the extracted maximum nodal accelerations are used to compute the weighted average maximum nodal accelerations in each direction due to each ground motion component. The weighting factors are the applicable nodal masses. Then in each direction the averaged maximum nodal accelerations due to the three components of earthquake motion are combined using the (1.0, .4, .4) rule as stated above. Table 3.7.2-27 and 3.7.2-28 show the worst case maximum ZPA accelerations for the EPGBs and the ESWBs, respectively. It is not clear how the maximum ZPA accelerations are used from the ten generic soil cases to determine member forces and moments and how the weighted average maximum nodal acceleration is calculated. The staff requests that the procedure and basis for calculating the weighted average maximum nodal accelerations be provided, as well as how the member forces and moments are determined once the maximum ZPA accelerations have been calculated from the ten soil cases.

Response to Question 03.07.02-34:

U.S. EPR FSAR Tier 2, Section 3.7.2.4.6 will be revised as follows:

> "These maximum nodal accelerations form the basis of the seismic loads used in the equivalent static analysis for the structural design."

For each of the three global directions, a weighted averaged maximum nodal acceleration for each major floor level is calculated as follows:

- Within the soil–structure interaction (SSI) analyses, maximum response nodal accelerations in x, y, and z directions are calculated at every node in the model for each of the 10 generic soil cases due to seismic excitation in x, y, and z directions, separately.
- Maximum nodal accelerations at each node are enveloped for all 10 generic soil cases. These enveloped nodal accelerations are plotted at the respective location of each node for each major floor level. These plots are referred to as bubble plots because, when printed, the acceleration at each node is represented by a circle. Circle size is proportional to acceleration magnitude, and circles within each floor level are plotted at the same relative locations as the locations of the nodes. Bubble plots visually indicate whether maximum nodal accelerations are uniform, or if there are regions of high and/or low accelerations within each floor level.
- Weighted averages of these enveloped maximum nodal accelerations are calculated for each floor level using nodal mass as the weighting factor. The resulting weighted average accelerations are nine components of responses in x, y, and z directions because of input motion in the x, y, and z directions.
- Co-direction responses in the global x, y, and z directions are then combined using the '1.0, 0.4, 0.4' method. The resulting combined of the enveloped weighted average maximum accelerations (CEWAMA) are in x, y, and z directions.

The static equivalent seismic loads are determined as follows:

- Starting from the above CEWAMA, the seismic loads are determined at each level of the building structure and they will be equal to or greater than the CEWAMA.
- Upon inspection on bubble plots of enveloped maximum nodal accelerations (for each floor elevation), if the responses within each floor elevation are relatively uniform, the seismic load for that elevation is the rounded-up value of CEWAMA. The average acceleration methodology is considered inadequate and is not used in regions that have a high response within each floor. In such cases, the seismic loads at all regions, including those with high response, are individually confirmed to have an adequately high seismic load.

Application of seismic loads to calculate member forces and moment:

- The above seismic loads are specified in terms of static acceleration at each major floor elevation. Three static analyses for seismic load in x, y, and z directions are performed. For each analysis, a static force (x, y, or z direction) is applied at each nodal point that is equal to the product of the total tributary mass at the node and the static acceleration of the seismic loads. These nodal forces are applied with positive sign for conservatism and the resulting member forces and moments are calculated from these static analyses.
- Perform load combination of the above static analyses, assuming seismic loads can be positive or negative.

The basis of using the seismic loads to calculate:

- The methodology of weighted average of maximum nodal acceleration using nodal mass as the weighting factor is consistent with conservation of total applied force. This methodology establishes that the combined product of weighted average maximum nodal acceleration and individual nodal mass for the entire floor elevation is the same as the combined product of the maximum nodal acceleration and individual nodal mass for the entire floor elevation. Therefore, if the weighted average accelerations are used, it is equivalent to using the maximum acceleration at each node multiplied by its nodal mass resulting in the same total inertia force and shear and overturning moment diagrams.
- At each floor elevation, the overall seismic loads are greater than or equal to the calculated weighted average values. Therefore, the seismic loads will produce exact or conservative results in story shear.
- Within each elevation, the distribution of inertia force is directly relative to the distribution of model mass.

FSAR Impact:

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

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and the 3D FEM's used in the static analysis. The modal time history analysis generates in-structure response spectra (ISRS) at representative locations of the structures for both the stick models and FEMs. The stick model is considered compatible with the FEM when the ISRS of the stick model are similar to those at corresponding locations of the FEM. For the NI Common Basemat Structures, computer codes used in such modal time history analyses are the GTSTRUDL code, Version 28, for the stick models and ANSYS code, Version 10.0, for the FEMs. For the NAB, the GTSTRUDL code, Version 29, is used in the modal time history analysis of both the stick model and FEM. The second application of the modal time history analysis is the local seismic analysis of the flexible slabs and walls in the NI Common Basemat Structures subsequent to the SSI analysis. In this case, the modal time history analysis of single-degree-of-freedom (SDOF) oscillators representing the flexible slabs and walls is performed to determine the amplified out-of-plane acceleration response and ISRS at such slabs and walls. The GTSTRUDL code, Version 28, is used in this application.

To solve Equation 1 numerically in the time domain using either the direct integration or modal superposition technique, the time step for numerical integration must be sufficiently small for stability and convergence of the solution. As a general rule, the value for the maximum time step is no larger than one-fifth of the lowest natural period of interest. Normally, the lowest period of interest need not be less than the reciprocal of the zero period acceleration (ZPA) frequency.

3.7.2.1.2 Response Spectrum Method

03.07.01-10

Response spectrum analyses are performed on flexible long span floors and roof of the NAB Non-Seismic Category I structure to obtain the amplified vertical accelerations of the floors.~~The response spectrum method is used in the local seismic analysis of certain slabs in the NAB to determine the out-of-plane seismic loads on the slabs.~~ Input motion to the analysis is the vertical ISRS at the slab locations generated from the seismic SSI analysis of the NI Common Basemat Structures and NAB.

Similar to the modal time history analysis method, when the response spectrum method is used it is assumed that the damping matrix [C] in Equation 1 may be explicitly represented by modal damping ratios so that the equation of motion given in Section 3.7.2.1 may be transformed to the equations of motion of the normal modes. The maximum seismic response of interest for each given mode is a function of the modal participation factor, mode shape and the input response spectrum acceleration at the corresponding modal frequency and damping ratio. The maximum modal responses are combined to determine the maximum response of interest in accordance with the combination method described in Section 3.7.2.7.

Table 3.7.1-1—Damping Values for Safe Shutdown Earthquake
Sheet 1 of 2

Item	Percent Critical Damping, SSE [4]
Reinforced concrete structures	7
Prestressed Concrete Structures	5
Welded Steel or Bolted Steel with Friction Connections [1]	4
Bolted Steel with Bearing Connections [1]	7
Motor, Fan, and Compressor Housings	3
Pressure Vessels, Heat Exchangers, and Pump and Valve Bodies	3
Welded Instrument Racks	3
Electrical Cabinets, Panels, and Motor Control Centers (MCC)	3
Piping Systems	
• Time history and ISM response spectrum analysis	~~4~~
• USM response spectrum analysis	<u>See Note 2</u>~~5~~ ~~[2]~~
• Systems susceptible to Stress Corrosion Cracking (SSC)	~~4 [2]~~
• Systems with supports designed to dissipate energy by yielding	~~4 [2]~~
Reactor Coolant System [6]	
• Component Shells	3
• Component Internals	4
• RPV Closure Head Equipment Tie Rods	7
• RCS Component Supports	4
• RCS Piping (including Surge Line)	4
• Fuel Assemblies [5]	30 max
Cable trays and supports [3]	
• Maximum Cable Loading [A, D]	10
• Empty [B, D]	7
• Sprayed-on Fire Retardant or other cable-restraining mechanism [C]	7
• Flexible Support Systems<u>[E]</u>	20 max
Conduits [3]	
• Maximum Cable fill [A]	7
• Empty [B]	5
HVAC Duct Systems	
• Pocket lock	10
• Companion angle	7
• Welded	4

03.07.01-15

Table 3.7.1-1—Damping Values for Safe Shutdown Earthquake
Sheet 2 of 2

Metal Atmospheric Storage Tanks • Impulsive Mode • Sloshing mode	 3 0.5

NOTES:

1. For steel structures with a combination of different connection types, use the lowest specified damping value, or as an alternative, use a "weighted average" damping value based on the number of each type present in the structure.

2. As specified in RG 1.61, Revision 1 and ANP-10264NP-A.~~Piping analysis using the USM response spectrum method and meeting the limitations specified in RG 1.61 is performed with damping of five percent of critical. The applicable limitations are summarized below.~~

   A. ~~Damping of five percent of critical is used completely and consistently.~~

   B. ~~Use of the specified damping values is limited only to response spectral analyses.~~

   C. ~~When used for reconciliation or support optimization of existing designs, the effects of increased motion on existing clearances and online mounted equipment should be checked.~~

   D. ~~Damping of four percent of critical is appropriate for analyzing the dynamic response of piping systems using supports designed to dissipate energy by yielding.~~

   E. ~~Damping of four percent of critical is applicable to piping in which stress corrosion cracking has occurred, unless a case-specific evaluation is provided on a case-by-case basis.~~

3. The following clarifications, taken from RG 1.61, are applicable.

   A. Maximum cable loadings, in accordance with the plant design specification, are to be utilized in conjunction with these damping values.

03.07.01-15

   B. Spare ~~cable tray and, initially empty~~and initially empty cable trays, may be analyzed with zero cable load and these damping values. (Note: Reanalysis is performed when put into service.)

   C. Restraint of the free relative movement of the cables inside a tray reduces the system damping.

   D. Selected damping value is to be justified and documented on an individual basis when cable loadings less than 50 percent of the maximum rated loading are specified for design calculations.~~When cable loadings of less than~~

~~maximum are specified for design calculations, justification of the selected damping value is performed on a case-by-case basis.~~

E. Higher damping values limited to flexibly supported rod and strut-hung trapeze systems, and strut-type cantilever and braced-cantilever cable tray systems loaded to greater than 50 percent of the maximum rated loading.

03.07.01-15

4. SSE damping values are used for generation of ISRS for the NI Common Basemat Structures. ~~OBE~~A damping value~~s are~~ of four percent is used for generation of the ISRS for the EPGB and ESWB.

03.07.01-12

5. The model elements representing the fuel assemblies are damped at a maximum of 30% per Framatome Topical Report BAW 10133PA-01 (including Addendum 1 and Addendum 2) (Reference 7).

6. Seismic analysis of the RPV Isolated model is by direct step-by-step integration time history analysis techniques, owing to the non-linear nature of the pressure vessel internals. As such, Rayleigh damping is applied. The Rayleigh mass and stiffness weighted damping coefficients are selected to provide generally conservative damping across the frequency range of interest, relative to the modal damping given in this table.

3A.3.2 Loads

The following loads are considered for the design of cable trays, conduits, and their supporting structures.

- Dead Load (D)—weight of cable trays or conduits, supports, cable inside of the raceways, tray covers, and other permanently attached components and fittings.
- Live Loads (L)—loads occurring during construction and maintenance. Live loads will not be less than a 250 lb load applied to a tray span in a manner providing worst case stresses in the tray and/or maximizing support loads. This load is not combined with seismic loads and is not applicable for conduits.
- Seismic (S)—See Section 3A.2.2.1.
- Thermal (T)—Loads resulting from thermal expansion or contraction. These loads are avoided by placing expansion/contraction joints along raceway runs.

3A.3.3 Load Combinations

Table 3A-4 lists the raceway and support loading combinations for the design of cable trays, conduits and supports.

3A.3.4 Allowable Stress Criteria

The basic stress allowables for carbon steel cold formed sections are in accordance with the AISI cold-formed structural design specification (Reference 4). The basic stress allowables for support structural steel, welds, and bolts are in accordance with Reference 8.

03.07.01-15

3A.3.5 Damping

~~The damping values for the design of cable tray and conduit systems are discussed in Section 3.7.1.2 and are contained inTable 3.7.1-1.~~The damping values for the design of cable tray and conduit systems and their associated supports are addressed in Section 3.7.1.2, and are provided in Table 3.7.1-1.

Cable trays with flexible support systems may use higher damping values based on testing, which includes the proposed installed configuration, loading, and support system. Historic tests have demonstrated that a substantial amount of energy is dissipated by friction between cables and through movement and bounding of cables within the tray. The increase in damping is more pronounced for loaded trays with higher input excitation but the maximum critical damping is limited to 20 percent for flexibly supported cable trays with a minimum loading of 50 percent of the trays full rated loading. Cable tray systems that are supported in accordance with the configurations described in ASCE 43-2005 are limited to a maximum critical damping of 15 percent for input ground motion ZPA limited to 0.25g. The damping values cable tray systems with less than 50 percent loading may be determined from Figure 3.7.1-16, which is dependent on the flexibility of the cable tray system, including both the cable tray and its supports, for an input ground motion ZPA up to and exceeding 0.35g. The damping value is to be reduced to the values indicated in Table 3.7.1-1 for conduit, cable trays loaded to less than 50 percent of the cable tray rated capacity, cable trays loaded primarily with conduit, or when rigid fire proofing materials are used causing the cables to become effectively bundled together.

03.07.01-15

3A.3.6 Seismic Analysis

~~The methods for seismic analysis are the same as described in Section 3A.2.4.4.~~This section describes the seismic analysis criteria for cable trays, conduits and their supports.

3A.3.6.1 Seismic Analysis Methods

Refer to Section 3.7.3.1.

3A.3.6.2 Determination of Number of Earthquake Cycles

Section 3.7.3.2 discusses the required number of earthquake cycles to be considered for seismic-induced fatigue. Rolled structural steel members for cable tray and conduit supports may be qualified for fatigue by evaluation in accordance with the provisions of ANSI/AISC N690 (Reference 8). Cold-formed members for cable tray and conduit supports may be qualified for fatigue by evaluation in accordance with the provisions of American Iron and Steel Institute (AISI), North American Specification for the Design of Cold-Formed Steel Structural Members (Reference 4). Connections for structural steel members are qualified by cyclic testing for the number of earthquake cycles specified in Section 3.7.3.2. Similarly, hardware components used to connect cold-formed members are also qualified by cyclic testing for the number of earthquake cycles specified in Section 3.7.3.2.

3A.3.6.3 Analytical Modeling Procedures

Refer to Section 3.7.3.3.

3A.3.6.4 Basis for Selection of Frequencies

Refer to Section 3.7.3.4.

3A.3.6.5 Analysis Procedure for Damping

Refer to Section 3.7.3.5 for analysis procedures for damping. The damping criteria is further described in Section 3A.3.5.

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.

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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.

4. P. Koss, "Seismic Testing of Electrical Cable Support Systems, Structural Engineers of California Conference," Bechtel Power Corporation, Los Angeles Power

3.7.2.1.3 Complex Frequency Response Analysis Method

With this analysis method, the damping of the system is not represented by the viscous damping matrix, [C], but as the imaginary part of a complex stiffness matrix. Thus Equation 1 becomes complex and must be solved in the frequency domain. To facilitate the analysis, the time history of input ground motion is transferred to the frequency domain by Fast Fourier Transform (FFT). The seismic responses calculated in the frequency domain are then transferred back to the time domain as outputs by inverse FFT.

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. For the SSI analysis results to be sufficient, the following requirements are met:

- A sufficiently high cut-off frequency is selected to ensure all significant SSI frequencies are included.
- A sufficient number of frequency points is used to accurately define the transfer functions within the cut-off frequency.
- The time step size for the input ground motion time histories is sufficiently small to be compatible with the selected cutoff frequency.

The SSI analysis generates the maximum ZPA at various floor locations, the floor acceleration time histories at representative locations for ISRS generation, the maximum member or element forces and moments, and the maximum relative displacements at the structural basemats with respect to the free-field input motions.

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The complex frequency response analysis method is also used in the soil column analysis using Bechtel computer code SHAKE2000, Version 1.1, to compute the free-field “in-column~~ground~~” motion at the foundation level of ESWB, for use as the input motion to the SSI analysis. This is because the SSI analysis of the ESWB considers structural embedment, and the input ground motion specified in Section 3.7.1 corresponds to a hypothetical free-field “outcrop” motion at the foundation level of ESWB. Bechtel code SASSI 2000 requires that the input motion, when specified at the foundation level, be an “in-column~~ground~~” motion converted from the “outcrop” motion through a soil column analysis.

3.7.2.1.4 Equivalent Static Load Method of Analysis

This analysis method is used to determine the seismic induced element forces and moments in the 3D FEMs of the NI Common Basemat Structures, EPGB, ESWB and NAB. In the analysis, equivalent static loads corresponding to the ZPAs generated from the seismic SSI analyses are applied to the 3D FEMs of the structure and basemat 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 Table 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.

3.7.2.4.7 Step 7 – Determining Amplified Seismic Responses for Flexible Slabs and Walls

(1) NI Common Basemat Structures

A concrete slab or wall in the NI Common Basemat Structures is considered flexible when the frequency of its first out-of-plane vibration mode is less than 40 Hz assuming uncracked concrete condition. Subsequent to the SSI analysis of the NI Common Basemat Structures, modal time history analyses using the GTSTRUDL code are performed for those SDOF oscillators simulating the first out-of-plane vibration mode of the flexible slabs or walls in the NI Common Basemat Structures. Input motions to the modal time history analyses are the floor acceleration time histories output from the SSI analyses at the applicable slab or wall locations. Since the flexible slabs and walls are reinforced concrete elements, the damping ratio of the SDOF oscillators is taken to be seven percent of critical.

Each flexible slab or wall in the NI Common Basemat Structures is assumed to respond in both un-cracked and cracked condition for out-of-plane vibration. The effective moment of inertia, I_c, of the slab or wall in cracked condition is taken to be 0.5 times the un-cracked, or gross, moment of inertia, I_g. Thus the out-of-plane vibration frequency of the cracked slab or wall is equal to 0.707 times that of the uncracked slab or wall. Generation of response spectra for the flexible slabs and walls in the NI Common Basemat Structures are discussed in Section 3.7.2.5.

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).

Table 2.1-1—U.S. EPR Site Design Envelope
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U.S. EPR Site Design Envelope	
Soil (Refer to Section 2.5)	
Minimum Static Bearing Capacity ~~(Static)~~	22 ksf in localized areas at the bottom of the Nuclear ksf on average across the total area of the bottom c basemat.
Minimum Dynamic Bearing Capacity	34.56 ksf at the bottom of the Nuclear Isla
Minimum Shear Wave Velocity (Low strain best estimate average value at bottom of basemat)	1000 fps
Liquefaction	None
Maximum Differental Settlement (across the basemat)	1/2 inch in 50 feet in any directi
Slope Failure Potential	No slope failure potential is considered in the design for U.S. EPR design certificatior
Maximum Ground Water	3.3 ft below grade
Minimum Coefficient of Static Friction (representative of soil basemat interface)	0.7
Inventory of Radionuclides Which Could Potentially Seep Into the Groundwater	
See Table 2.1-2—Bounding Values for Component Radionuclide Inventory	
Flood Level (Refer to Section 2.4)	